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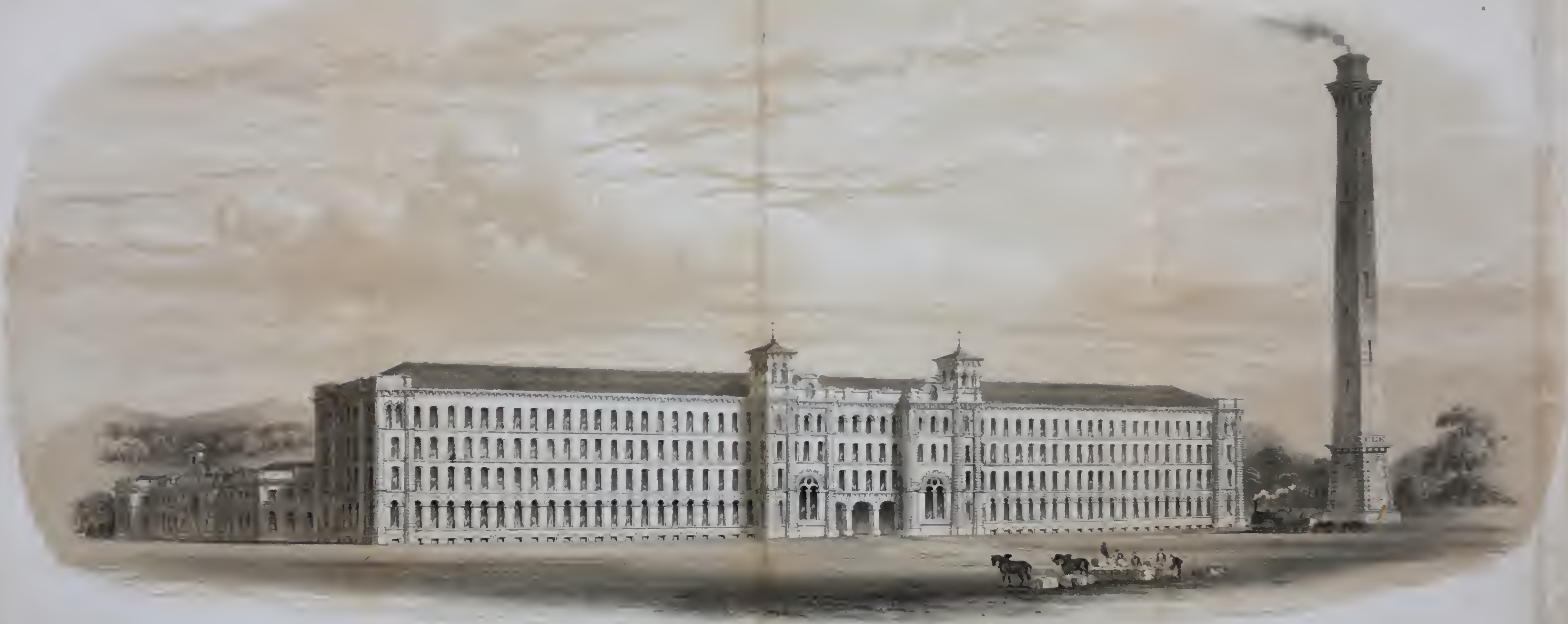

SALTAIRE MILLS NEAR BRADFORD,
THE PROPERTY OF TITUS SALT ESQ^{RE}

Art Arch. B

J. Macdougall.
Dec. 2 64

ON

THE APPLICATION

OF

CAST AND WROUGHT IRON

TO

BUILDING PURPOSES.

BY

WILLIAM FAIRBAIRN, C.E. F.R.S. F.G.S.

PRESIDENT OF THE MANCHESTER LITERARY AND PHILOSOPHICAL SOCIETY; CORRESPONDING
MEMBER OF THE INSTITUTE OF FRANCE; MEMBER OF THE ROYAL ACADEMY OF
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WITH ADDITIONS, &c.

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1864.

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TO

SIR DAVID BREWSTER, K.H. LL.D. F.R.S.L. & E.

Hon. M.R.I.A. F.G.S. F.R.A.S.
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PRINCIPAL OF THE UNIVERSITY OF EDINBURGH,
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AS A MEMORIAL OF LONG AND CORDIAL FRIENDSHIP,

AND

AS A PUBLIC EXPRESSION OF

ADMIRATION OF HIS EMINENT SCIENTIFIC ATTAINMENTS,

THIS VOLUME IS DEDICATED

BY

THE AUTHOR.

PREFACE

TO

THE FIRST EDITION.

In the following pages I have endeavoured to collect the sum of our practical knowledge on the use of iron, in its combination with other materials, in the construction of fire-proof buildings. The subject is one of vast public importance; and although I am conscious that in its investigation I have laboured under many disadvantages, occasioned by the frequent interruptions of professional engagements, and have not been able, in the limited time at my disposal, to enter as much into detail as I could have wished,—yet, if I have succeeded in shaping information into such form that the engineer, architect, or builder can consult it with facility and profit, I shall have accomplished the end I had in view. It is undeniable that great want of judgment has been displayed in many examples of buildings of even very recent date; and it is to be lamented that so much ignorance of those undeviating laws which govern the strength of materials should still prevail. Experimentalists and mathematicians have provided the knowledge; but practitioners, I fear, have in a great degree failed to avail themselves of it.

In the remarks on cast-iron beams, I have been much assisted by the labours of Watt, Tredgold, Dulong, and Barlow, and by the more recent and conclusive experiments of Professor Hodgkinson, made at my establishment some few years ago. The question of the strength of trussed girder beams has long been one of doubt and difference of opinion; but the experiments recorded in the following pages, aided by the able theoretical investigation of my friend, Mr. Tate, are conclusive as to their insufficiency, and the exceedingly injudicious distribution of material in that form.

I conceive the most important section of the book will be that which is given to a recommendation of the advantages of wrought-iron beams or joists, in substitution of the more cumbersome and uncertain ones of cast iron now in general use. To this question I have devoted especial attention, and have endeavoured to apply the numerous experiments with which for some years past I have been engaged to the development of a principle which, if judiciously carried out, will lead to important changes, both of an economical and secure character, in the construction of fire-proof structures. The perfect efficiency of malleable iron beams is proved by numerous examples; and, in my opinion, a more extensive adoption of them is alone wanting to excite the attention of the iron manufacturer, and induce that application of talent and capital which would speedily reduce the cost of production.

I have devoted a separate section to a brief description of the magnificent establishment, approaching completion, at Saltaire. The vastness of its conception, and the completeness of its finish, render the design of Mr. Salt an honour to his name; and it has been a source of pride to myself to close the active service of a professional life in connexion with so important and so honourable an undertaking.

W. F.

Manchester: 1854.

PREFACE

TO

THE SECOND EDITION.

THE practical value of this Work has been evinced by the manner in which it has been received by the public. In a period of less than two years the whole of the copies first printed were sold, and I have been induced, at the earnest request of the publisher, to revise and enlarge the volume in a second edition.

In attempting to fulfil this duty, I have endeavoured, whenever possible, to support the conclusions I had arrived at by direct experiment, and in cases where that was wanting, I have relied on the exercise of a judgment guided by lengthened experience in similar constructions.

In the section on Cast-iron Beams there was little to add to what was contained in the first edition, beyond some tables of the results of experimental researches into the strength and constitution of iron, and its improvement by certain processes of manufacture. In the course of my experience I have seen no reason to change the opinions I had previously expressed on the uncertain character of trussed girders, and I still advise the exercise of great caution in their employment, from the conviction that they are equally unsound in principle and dangerous in practice. In the section on trellis beams some additional examples have been added.

In order to make the work more widely useful and complete, I have added the entire section on Wrought-iron Bridges, containing such results of experimental research as are applicable to these constructions, also the mathematical formulæ deduced from them; rules for calculating the strength and proportioning the parts, and examples of works either erected or now in course of construction. Conceiving that the plans and general outline of designs for crossing the Rhine at Cologne with a wrought-iron bridge—which, although fully prepared, were not carried out—might be useful in the future designing of works of that magnitude, I have ventured to give a brief account of the origin and progress of that project, and of the delays, intrigues, and other disingenuous contrivances which led to its suppression, and the adoption of the structure now in course of erection.

Several errors which had crept into the former edition have been rectified, and I hope that the same favourable consideration which was so liberally extended to its predecessor may be granted to the present edition.

W. F.

Manchester: *October* 1857.

PREFACE

TO

THE THIRD EDITION.

THE increasing demand for this Work is the best evidence of its utility. It has been out of print ever since the death of the late Mr. Weale, under whose auspices the first and second editions were published. I have, however, made arrangements with my publishers, Messrs. Longman & Co., for another Edition, in a form corresponding with my other works. In this Edition I have endeavoured to bring the subjects on which the volume treats down to the present state of our knowledge of constructive art, and I have introduced several additions and improvements that have occurred since the last publication.

It will be observed, when treating of Fire-proof Buildings, that wrought-iron beams and girders are to some extent superseding the use of cast-iron, and in that section will be found examples upon a scale sufficiently large to ensure increased security for the support of arches, or any other description of flooring which the character of the building may require. In this department, it is however to be regretted, that architects do not avail themselves of a material so admirably adapted as wrought iron is for strength and security from fire, to a much greater extent than has hitherto been adopted. The greatest number of our dwelling-houses and public buildings are not strictly fire-proof, but only partially so; and instead of making the whole free from risk, we appear to be content if the main beams of the different flats are of iron, whilst the floors, joists, &c., are chiefly composed of wood.

This appears to be an omission to which I am desirous to direct attention, and I trust the time is not far distant when the extended distribution of wrought iron in all its forms and appliances may be calculated upon. Rolled iron joists—similar to those so extensively used by French architects—would render the whole construction fire-proof, save a high rate of insurance, and enhance the value and permanency of the structure. Examples have already been given for these appliances, and I would, in addition, refer to a recent erection, eight stories high, for a sugar refinery, where the whole of the main beams are of wrought iron, supporting brick arches, with heavy loads on the floors above.

The section on Wrought-iron Bridges has been enlarged with additions and examples, calculated to determine the value of the varied and different forms now in use. Those on the Lattice principle I have investigated experimentally, and in this Edition will be found formulæ from which to calculate the position and strains on the struts and tension bars of these important structures. It was my intention to have given a series of experiments on homogeneous iron and steel as applied to beams and bridges, but the manufacture, although greatly improved, is not yet sufficiently advanced, to insure safety in the use of this new material. This does not arise so much from want of knowledge of its mechanical properties and chemical compounds, as from want of uniformity in the manufacture. I, however, entertain hopes that the same devoted attention to the subject as hitherto has been pursued for the last few years, will overcome every difficulty, and produce a material of greatly-increased strength, and on which the architect and engineer may safely depend.

W. F.

CONTENTS.

PART I.

PAGE
ON CAST-IRON BEAMS FOR SUPPORTING THE FLOORS OF BUILDINGS . 1
Historical Notice of Cast-iron Beams with Flanches . . 5
Experiments made at Leeds and Bradford by the Author . 6
Tredgold's Experiments 8
Experiments made by Mr. Hodgkinson and the Author at his Works in Manchester 9
Rules for the Strength of Cast-iron Beams 13
Table of Results 25

ON COMPOUND OR TRUSSED CAST-IRON BEAMS OR GIRDERS . . 27
Approximate Rule for calculating the Strength of a Truss-Beam 36
Comparison of Cost 37
Trussed Beams where the Rods are attached to Points above the Line of the top Flanch 38
Experiments to determine what Advantages, if any, are derived from Wrought-iron Trusses applied to Cast-iron Beams as an additional Support 40
Summary of Results 43
General Remarks relative to Cast-iron Beams . . . 44
On the Influence of Time and Temperature on the Strength of Cast-iron Bars 47
Tables of Experiments on Rectangular Bars relative to the Effects of Time 48
On the Effects of Temperature 54
On the comparative Strength of Iron after successive Re-meltings 56
Mixtures of Cast Iron 62
Process of toughening Cast-iron — Experiments — Cupola — Air-furnace 64
Experimental Inquiry on the Transverse, Tensile, and Compressive Strength of Cast-iron 74

PART II.

PAGE
ON WROUGHT-IRON BEAMS FOR SUPPORTING THE FLOORS of BUILDINGS, AND FOR OTHER PURPOSES 78
Experiments on the Strength, &c., of Wrought-iron Beams . 89
ON WROUGHT-IRON TRELLIS GIRDERS 114
Experiments to determine the Strength and Security of open Trellis Girders 116

PART III.

ON THE CONSTRUCTION OF FIRE-PROOF WAREHOUSES . . . 137
THE SALTAIRE MILLS 177
Experimental Test of Beams 181

PART IV.

THE ADAPTATION OF MALLEABLE-IRON BEAMS OR GIRDERS FOR THE CONSTRUCTION OF BRIDGES 187
Timber 190
Stone 193
Brick 198
Wrought and Cast Iron 199
Introduction of Iron Bridges 200
Lattice and Warren Bridges 206
Tubular and Tubular Girder Bridges 213
Strength and Proportions of Tubular Bridges . . . 214
Details of Spey Bridge 254
Plate Bridges 262
Strains in the Interior of Beams 268
Bridges over the Rhine at Cologne. 275

APPENDIX.

I. Remarks relative to Wrought-iron Beams 287
II. Report on the Causes of the Fall of the Cotton Mill, at Oldham, in October, 1844 287
III. On some Defects in the Principle and Construction of Fire-proof Buildings 293
IV. The Box and the Plate Beam 298
V. The Cologne Bridges 300
INDEX 305

LIST OF PLATES.

PLATE I. Saltaire Mills, near Bradford . . . *Frontispiece*
„ II. Ground-plan of Mill at Saltaire . .
„ III. Section through Mill—Section and Elevation of Hollow Brick—Section of Fireproof Arching . . . . .
„ IV. Ground-plan of part of the Rhine between Cologne and Deutz . . . .
„ V. Elevation and Plan of Proposed Tubular and Girder Bridge for crossing the Rhine at Cologne . . . .
„ VI. Elevation and Plan of Proposed Tubular Bridge across the Rhine at Cologne
To be placed at end of Work

EXPERIMENTAL RESEARCHES.

PART I.

ON CAST-IRON BEAMS FOR SUPPORTING THE FLOORS OF BUILDINGS.

The exact time at which cast iron came into use appears to be very uncertain ; but we read of its application for casting cannon shortly after the invention of gunpowder. During the days of Savery and Newcomen it was partially used in the construction of their steam-engines and pumps; and shortly after Newcomen's invention his cylinders were made of it. Its value was also appreciated at an early period by Smeaton, who, according to Tredgold, combated the prejudices against it, upwards of a hundred years ago, in the following language:—

'If the length of time of the use of these cast-iron utensils is not sufficient, I must add, that in the year 1755, that is, twenty-seven years ago, for the first time I applied them as totally new subjects, and the cry then was, that if the strongest timbers are not able for any great length of time to resist the action of the powers, what must happen from the brittleness of cast iron? It is sufficient to say that not only those very pieces of cast iron are still at work, but that the good effect has in the north of England, where first applied, drawn them into common use, and I never heard of one failing.'

At the time Smeaton wrote, the art of casting in iron was very imperfect, and we have seen to what varied and extended uses it has since been applied. The resisting powers of the material, however, have not increased with the extent of its application. On the contrary, I much fear that it has deteriorated in quality; not from want of knowledge in the process of smelting, or skill in the treatment of the ores, fuel, &c., but simply from a desire to lessen the cost of production. Cast iron may doubtless be obtained of great strength and purity at the present day, but these qualities are often accompanied by others, which render its application in beams and other heavy structures hazardous. Smeaton, Wilkinson,* Watt, Rennie, Murdock, Telford, and other celebrated engineers, materially improved the art of casting, and extended its application to the steam-engine, mill-work,† bridges, and machines.

The first instance on record of the successful application of cast-iron beams to the purposes of building is that of a fire-proof cotton-mill, erected by Messrs. Philips and Lee of Manchester. This mill was built in the year 1801; the iron beams and columns were designed by Messrs. Boulton and Watt, and were of the following sectional dimensions at the middle.

This beam (fig. 1) was the first of the kind made, and, considering the limited state of our knowledge at that period, it reflects great credit upon the skill of the designer. If we apply

---

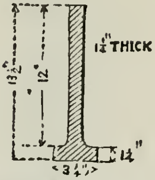
* 'One of the boldest attempts with a new material was the application of cast iron to bridges. The idea appears to have originated, in the year 1773, with the late Thomas Farnolls Pritchard, then of Eyton Turret, Shropshire, architect, who, in communication with the late Mr. John Wilkinson, of Broseley and Castlehead, iron-master, suggested the practicability of constructing wide iron arches, capable of admitting the passage of a river, such as the Severn, which is much subject to floods. This suggestion Mr. Wilkinson considered with great attention, and at length carried into execution between Madeley and Brosely, by erecting the celebrated iron bridge at Colebrook-dale, which was the first construction of that kind in England, and probably in the world. This bridge was executed by a Mr. Onions, with some variations from Mr. Pritchard's plan, under the auspices and at the expense of Mr. Darby and Mr. Reynolds, of the iron-works at Colebrook-dale. Mr. Pritchard died in October 1777. He made several ingenious designs, to show how stone or brick arches might be constructed with cast-iron centres, so that the centre should always form a permanent part of the arch. These designs are now in the possession of Mr. John White of Devonshire Place, one of his grandsons, to whom I am indebted for these particulars.'—TREDGOLD *On the Strength of Cast Iron.*

† On the occasion of a visit to Soho, near Birmingham, upwards of thirty years ago, Mr. W. Murdock showed me one of the first bevel-wheels cast from iron. It supported a sun-dial in the front of his house; and I believe he stated it to have been cast in Ayrshire, from patterns made either by himself or by his father, who was a miller and millwright.

Mr. Hodgkinson's rule to it, we shall find that, in the absence of experiment, Watt had made a tolerably correct approximation to the true proportion of the parts of the beam, so as to secure a maximum strength with a given quantity of material.*

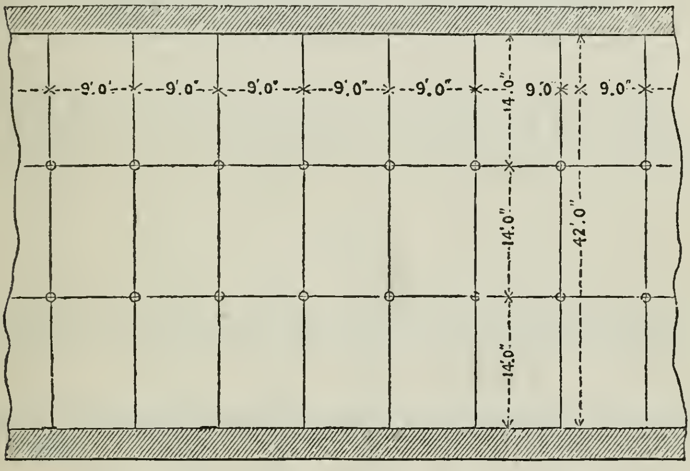
The mill is a large building of about 140 feet long, 42 feet wide, and seven stories high. It contains about 648 square yards on each floor; and the iron beams which extend across the building, from wall to wall, at regular distances of 9 feet, are divided into three lengths (A, A, and B), as shown on the plan and section, figs. 3 and 4.

Fig. 1.



Whole area, 19·05 inches
Bottom flange, 4·06 ,,

Fig. 3.



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* If we take the area of the bottom flange to be 4·06 inches, the depth of the beam in the middle 13¼ inches, and the distance between the supports 14 feet, then, by Mr. Hodgkinson's formula, we have $w = \frac{26 \times 4·06 \times 13·25}{168} = 8·32$ tons = the breaking weight in the middle. But assuming that the same quantity of metal was run into the form of greatest strength, as at A, fig. 2, we shall then have an area for the bottom flange = 7·5 inches, which gives $w = \frac{26 \times 7·5 \times 13·25}{168} = 15·3$ tons, nearly; that is, a breaking weight nearly double that of the original beam. It is probable, however, that

Fig. 2.

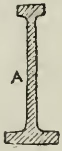
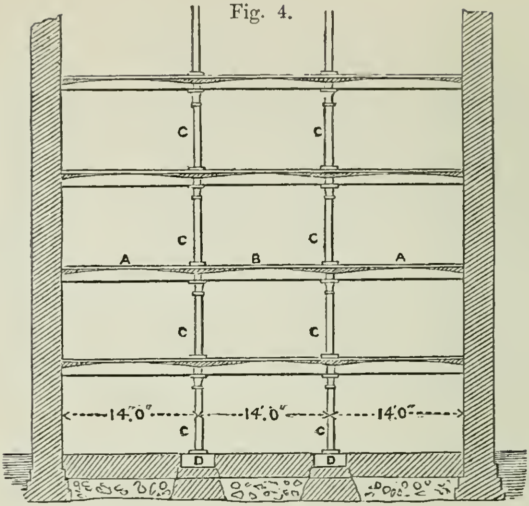
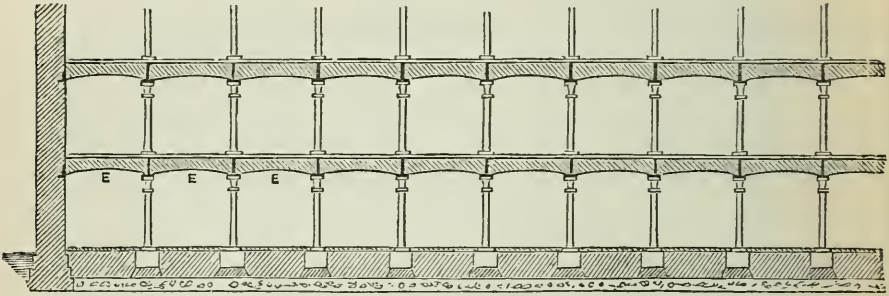


Fig. 4.



The following woodcut (fig. 5) exhibits a longitudinal section of portions of the basement and first stories of the mill, with sections of the iron beams and arches. The arches E, E, E,

Fig. 5.



&c., are 9 inches in depth at the springing, $7\frac{1}{4}$ inches at a short distance on each side, and a half brick, or $4\frac{1}{2}$ inches, in the

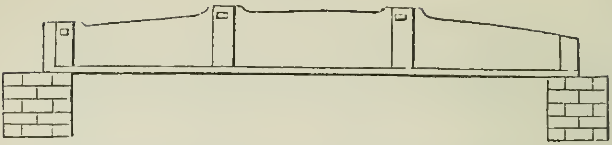
Messrs. Boulton and Watt's beam would carry upwards of 10 tons, owing to the greatly increased thickness of the vertical part of the beam, which, it will be observed, is nearly double that of a beam of maximum form and strength.

middle. Altogether the experiment—considering the construction of such buildings at that time—was eminently successful, and became the pioneer of that system of fire-proof structure which now distinguishes the manufacturing districts of this country.

From 1801 till 1824 little or no variation took place in the form of beams, and for a quarter of a century Messrs. Philips and Lee's mill offered the model for similar buildings. In the year 1827 Mr. Hodgkinson commenced his well-known inquiry into the strength of iron beams at my works in Manchester; and for many years afterwards he carried on these and other experiments with great success. The economising of material was of such evident importance, and the experimenter's views were so rational, that the requisite means for the most thorough investigation of the subject were at once placed at his disposal. The results of these experiments are embodied in the valuable papers which have appeared in the *Memoirs of the Manchester Philosophical Society, Second Series*, vol. v.

Previously to Mr. Hodgkinson's investigation I had the construction of several extensive fire-proof buildings, one for Messrs. Gott of Leeds, and another for Mr. Wood of Bradford; and entertaining doubts as to the security of the cast-iron beams, a series of experiments was made on a large scale, in order to give confidence in the safety of the constructions. The results of these experiments, which are here inserted, showed that the area of the bottom flange should be increased, in order to obtain a stronger and better sectional form than the one used by Boulton and Watt at Philips and Lee's mill. In making the experiments, I found it necessary, in order to arrive at correct results, to ascertain the deflections of the beam as the weights were laid on: this was accomplished by piling dead weights on a platform, suspended from the centre of the beam. This platform was attached to a strong shackle with an open side, for the purpose of removing and admitting the beam; and thus, by a saddle and screw, the weights were lowered or raised at pleasure. The following figure (fig. 6) represents a side view of the beams, the ribs at the bottom of which were, as usual, of uniform width and thickness.

Fig. 6.

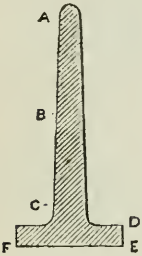


Experiments made at Leeds in 1824.

EXPERIMENT I.

Distance between the supports, 14 feet.
Depth of beam in the middle, 15 inches.
Depth of beam near the ends, 9½ „

Fig. 7.



Weight of beam (taken from the average weights of several beams from the same model), 7 cwt. 3 qrs. 20 lbs.

Dimensions of section (fig. 7).

Thickness at A = $\frac{5}{8}$ inch.
„ C = 1 „
„ DE = 1 „
„ FE = 5 „

Weights.	Deflections in parts of an inch.	Remarks.
4 tons. 10 cwt.	. 0·21	
6 „ 0 „ . .	·278	
10 „ 0 „ . .	·48	A little warped.
11 „ 4 „ . .	·537	The top edge of beam pressed considerably out of perpendicular.
12 „ 10 „ . .	·665	The pressure outward much increased, and danger of breaking.

The experiments on this beam were not performed with the same accuracy as those subsequently made at Leeds and Bradford; it was nevertheless tested with considerable care.

EXPERIMENT II.

Distance between supports, 16 feet.
Depth of beam in middle, 15 inches.
Depth of beam near ends, 10 „

Dimensions of section (see fig. 7).

Thickness at A = $\frac{7}{8}$ inch.
„ C = 1$\frac{3}{8}$ inches.
„ DE = 1$\frac{3}{8}$ „
„ FE = 6 „

Weights.	Deflections in parts of an inch.
6 tons. 0 cwt.	0·2
8 „ 5 „	·28
11 „ 0 „	·4
13 „ 5 „	·475
16 „ 0 „	·55
18 „ 5 „	·675
21 „ 0 „	·85
23 „	Broke, after sustaining the weight two hours.

Four other beams were tested; they were, however, more or less imperfect, and broke two or three feet from the centre, where the flaws happened to be.

Experiment made at Bradford in 1825.

Distance between supports, 20 feet 9 inches.
Depth of beam in middle, 18 inches.
Depth of beam near ends, 11$\frac{1}{2}$ „

Dimensions of section in inches.

Thickness at A = 1 inch.
„ C = 1$\frac{1}{2}$ inches.
„ DE = 1$\frac{1}{2}$ „
„ FE = 6 „

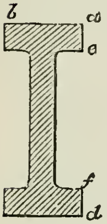
Weights.	Deflections in parts of an inch.
13 tons	1·16
18 „	1·25
19 „	Broke, after sustaining the weight some time.

The experiments here recorded, taken in connection with those subsequently given in Mr. Hodgkinson's paper, may probably be considered as the first attempts to improve the design of cast-iron beams, since their first application in Philips and Lee's mill.

About this time (1824) Tredgold published the second edition of his work on the Strength of Cast Iron; and his experiments on the transverse strength of that material must have been made two or three years previously. The only experiment then made by Mr. Tredgold which at all bears on the present inquiry is that recorded in Experiment I., which, for the sake of comparison, is here inserted.

'A joist of cast iron, of the form described in fig. 8, was submitted to the following trials. It was placed on its edge, and supported at the ends only, the distance between the supports being 19 feet. The deflection from its own weight was three-fortieths of an inch.

Fig. 8.



'When it was laid flatwise, the deflection from its own weight was 3·5 inches, the distance of the supports remaining 19 feet.

'The whole depth, $a\,d$, was 9 inches; the breadth, $a\,b$, 2 inches; the depth of the middle part, $e\,f$, $7\frac{1}{2}$ inches; and the breadth of the middle part three-fourths of an inch.

'It may be easily shown, that to derive the value of a from the experiment on the edge, we may use an equation of this form:

$$a = \frac{40\,\mathrm{B}\,\mathrm{D}^3\,d(1-p^3q)}{\frac{5}{8}\,\mathrm{W}\,\mathrm{L}^3} = \frac{64\,\mathrm{B}\,\mathrm{D}^3 d(1-p^3q)}{\mathrm{W}\,\mathrm{L}^3};$$

in which D is the whole depth, and $p\,\mathrm{D}$ the depth of the middle part, and B the whole breadth, and $q\,\mathrm{B}$ the breadth, after deducting that of the middle part.

'In our experiment $\mathrm{D} = 9$ inches, and $p\,\mathrm{D} = 7{\cdot}5$, or $p = {\cdot}833$. Also, $\mathrm{B} = 2$ inches, and deducting three-fourths, the breadth of the middle, we have $q\,\mathrm{B} = 1{\cdot}25$, or $q = {\cdot}625$. And the weight of the part of the joist between the supports being 540 lbs., we find $a = {\cdot}00124$.

'The equation for finding the value of a, in the experiment with the joist, flatwise, is

$$\frac{64\;\mathrm{B}\,\mathrm{D}^3 d\,(1+p^3q)}{\mathrm{W}\;\mathrm{L}^3} = a = {\cdot}00092,$$

where

$$\text{D}=2 \text{ inches}, \text{B}=9-7{\cdot}5=1{\cdot}5, p=\frac{{\cdot}75}{2}, \text{ and } q=\frac{7{\cdot}5}{1{\cdot}5}.$$

'I consider the value of a, derived from the experiment with the joist flatwise, as nearest the truth, because the deflection was so considerable, that a small error in measuring it would not sensibly affect the result, whilst there must be some uncertainty in ascertaining so small a deflection as three-fortieths of an inch in 19 feet; and a very small error in this measure would cause the difference between the results. I have, however, given it as I determined it at the time, and the manner of calculation may be useful in other cases. If the mean be taken between the results, it is

$$\frac{{\cdot}00124+{\cdot}00092}{2}={\cdot}00108.$$

In the experiment, flatwise, we obtain a constant multiplier extremely near to that determined from a bar of the same iron an inch square and 34 inches long, and it differs only about one-twelfth part from the one employed for calculating the table.'

It is to be regretted that Mr. Tredgold did not break this beam, instead of simply ascertaining the deflection from its own weight. As Mr. Tredgold adopted the beam with equal flanges, top and bottom, and recommended it as the strongest and best form for supporting the floors of buildings, it will be necessary for me to show the inaccuracy of such a conclusion, and to give the progressive improvements that have since been made, by introducing a few of the more interesting experiments of Mr. Hodgkinson,* to whom science and the public are assuredly indebted for the section of greatest strength.

EXPERIMENT I.†

Beam with equal rib at top and bottom.

Distance between supports, 4 feet 6 inches. Depth of beam $5\frac{1}{8}$ inches.

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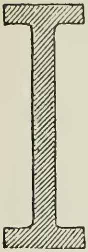
* The extracts from Mr. Hodgkinson's paper, in the Memoirs of the Manchester Philosophical Society, are embraced between pp. 9 and 24.

† All the sections in these experiments are drawn to one-fourth their real lineal dimensions.

Fig. 9.

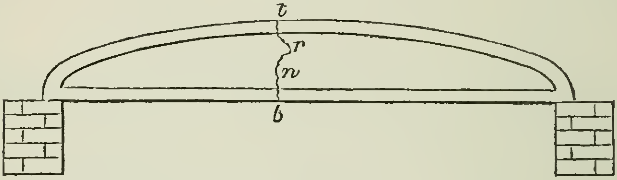


Dimensions of cross section, at place of fracture, in inches and parts.

Area of top rib . . . . $=1{\cdot}75 \times {\cdot}42 =$ ·735
Area of bottom rib . . $=1{\cdot}77 \times {\cdot}39 =$ ·690
Thickness of vertical part, between ribs = ·29
Area of above section . . . . = 2·82
Weight of casting . . . . $=36\frac{1}{4}$ lbs.
Breaking weight = 6678 lbs. = 59 cwt. 70 lbs.

The form of fracture is represented by the line $b\ n\ r$ (fig. 10), where $t\,r = {\cdot}6$, and $b\,n = 2{\cdot}5$, the figure being a side view of the beam.

Fig. 10.



To find the strength per inch of cross section, we have, dividing the breaking weight by the area, $\frac{6678}{2{\cdot}82} = 2368$ lbs. per inch. As this quantity in each beam may be taken as an index of its strength, we shall use it to compare the strengths of those beams which are of the same length and depth, which is the case in the following experiments.*

---

* From the author's Treatise on Tubular Bridges, p. 280, we have

$$\mathrm{W} = \frac{\mathrm{A}d\mathrm{C}}{l} \ldots (1),$$

where A is put for the area of the section of the material in square inches, d the depth in linear inches, l the distance between the points of support in linear inches, and C a constant determined by experiment for the particular form of the tube.

Hence we find

$$\mathrm{C} = \frac{\mathrm{W}l}{\mathrm{A}d} \ldots (2).$$

The value of C, determined for different forms of beams, gives us their comparative strength.

Now, for beams of the same length and depth, we have

$$\mathrm{C} = \frac{\mathrm{W}}{\mathrm{A}} \ldots (3),$$

that is, the comparative strength of beams of this description is found by dividing their breaking weights by their sectional area.

Comparing this with the result from Experiment IV., where the beam bore 2584 lbs. per inch, we find $2584-2368=216$ $=$defect.

∴ loss in strength $=\frac{216}{2584}=\cdot 083$ or $\frac{1}{12}$ nearly, in parts of what the common beam bore.

This is essentially the form of section which Mr. Tredgold has represented to be that of the strongest beam, whilst the elasticity is perfect. Our future experiments will sufficiently show that this is not the case.

Experiment II.

Beam with areas of section of top and bottom rib as 1 to 2.

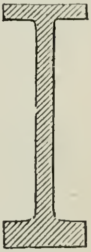
Distance between supports, 4 feet 6 inches. Depth of beam, $5\frac{1}{8}$ inches.

Dimensions of cross section.

Area of top rib . . $=1\cdot 74\times\cdot 26=$ ·45 inches.
Area of bottom rib. $=1\cdot 78\times\cdot 55=$ ·98 „
Thickness of vertical part . . $=$ ·30 „
Area of cross section . . . . $=2\cdot 87$ „
Weight of casting . . . . . . $=39$ lbs.
Breaking weight = 7368 lbs. = 65 cwt. 88 lbs.

It broke obliquely about four inches from the middle, the top inclining to it.

Fig. 11.



The form of fracture at the top of the beam was nearly the same as in Experiment I.; here $t\,r=\cdot 55$ inches: see second figure to that experiment.

To find the strength per inch of section, as in the last experiment, we have $\frac{7368}{2\cdot 87}=2567$ lbs. per inch. Comparing this with the result of Experiment IV., we find $2584-2567=17=$ defect.

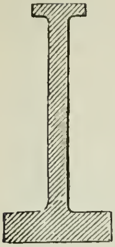
$$∴ \text{ loss in strength} = \frac{17}{2584} = \cdot 0066 \text{ or } \frac{1}{152}.$$

Experiment III.

Beam with top to bottom rib as 1 to 4.

Distance between supports, 4 feet 6 inches. Depth of beam $5\frac{1}{8}$ inches.

Fig. 12.



Dimensions of cross section in inches.

Area of top rib . . . . = 1·07 × ·30 = ·32
Area of bottom rib . . . = 2·1 × ·57 = 1·2
Thickness of vertical part . . . . . = ·32
Area of cross section . . . . = 3·02
Weight of casting . . . . = 40 lbs.
Ultimate deflection, upwards ¼ of an inch.

Breaking weight = 8270 lbs. = 73 cwt. 94 lbs. It broke nearly in the middle.

Dividing the breaking weight by the area gives the strength per inch of section $=\frac{8270}{3\cdot02}=2737$ lbs. But Experiment IV. gives 2737 lbs. per inch.

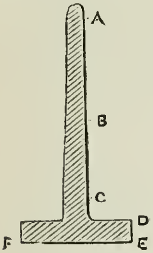
Hence $2737-2584=153=$ excess.

$$\therefore \text{ gain in strength } 153=\frac{1}{17} \text{ nearly.}$$

EXPERIMENT IV.

Beam cast in the common form from Messrs. Fairbairn and Lillie's model. Distance between supports and depth of beam as before.

Fig. 13.



Dimensions of section in inches.

Thickness at A = ·32
" B = ·44
" C = ·47
" FE = 2·27
" DE = ·52
Area of Section = 3·2 inches.
Weight of casting = 40½ lbs.

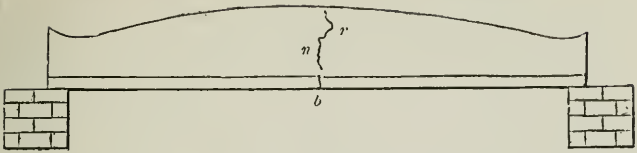
Deflection with	5758 lbs.	·25 "
"	7138 lbs.	·37 "

Breaking weight = 8270 lbs.

The beam twisted a little before breaking: this is not usually the case in the other beams from the same model.

Form of fracture as in figure, $tr = \cdot75$.

Fig. 14.



$$\text{Strength per inch of section} = \frac{8270}{3{\cdot}2} = 2584 \text{ lbs.}$$

All the preceding experiments were made on beams cast *on their side* from iron, of which the following is a description.

MIXTURE.

$\frac{1}{3}$ of Blaina, No. 2, } Welsh.
$\frac{1}{3}$ of Blaina, No. 3, }
$\frac{1}{3}$ of wss, No. 3, Shropshire.

This mixture is a strong iron, and therefore well suited for beams.

In the following experiments the beams were cast *erect*, but upside down, as there is an accession of strength from that cause.

EXPERIMENT IX.

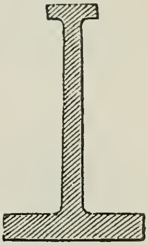
Ratio of the ribs, 1 to $4\frac{1}{2}$ nearly.

Distance between supports and depth, as before.

Dimensions of section in inches.

Area of top rib . $=1{\cdot}05 \times {\cdot}34 = 0.357$
Area of bottom rib $=3{\cdot}08 \times {\cdot}51 = 1{\cdot}570$
Thickness of vertical part . $= {\cdot}305$
Area of section . . . . . $=3{\cdot}37$ inches.
Weight of beam . . . . . $= 44\frac{3}{4}$ lbs.
Breaking weight, 10727 lbs. = 95 cwt. 87 lbs.

Fig. 15.



It broke by tension, 4 inches from the middle, but slanting towards it; and there seemed to be a small flaw in the bottom rib at the place of fracture. Here $t\,r = {\cdot}6$ inch (see fig. 10).

$$\text{Hence strength per inch of section} = \frac{10727}{3{\cdot}37} = 3183 \text{ lbs.}$$

Comparing this with the result of Experiment X., gives $3183 - 2792 = 391 =$ excess.

$$\therefore \text{gain in strength} = \frac{391}{2792} = \tfrac{1}{7} \text{ nearly.}$$

Remark. Though this beam had a larger bottom rib, it nevertheless broke by tension, or by tearing the bottom part first, which was evident, as it had neither been crushed nor broken by a wedge. This I had noticed to be the case in every experiment. There had been gained $\frac{1}{7}$ in strength, above that of the common beam, by the addition already made; and it was probable we might add still more to the lower rib without danger of fracture by compression; for in no case, except of the common beam, which sometimes twisted before it broke, had there been the slightest appearance of over-compression. This idea will be pursued in our future experiments.

Experiment X.

Common beam, cast upside down, in the usual manner.

This, like the rest, was from the same model as that in Experiment IV.

Distance between supports as before.

Dimensions of section in inches. (See fig. 13, Experiment IV.)

Thickness at A = ·29
,, B = ·425
,, C = ·46
,, F E = 2·3
,, D E = ·53
Area of Section = 3·16 inches.
Weight of beam = 40½ lbs.
Breaking weight = 8823 lbs.*

It broke 1½ inches from the middle. The form of fracture was nearly as in Experiment IV.; here $b\,n = 2{\cdot}25$ and $t\,r = {\cdot}8$ (see fig. 12).

Hence strength per inch of section $= \frac{8823}{3{\cdot}16} = 2792$ lbs.

In the following experiments, the bottom rib is considerably increased, agreeably to remarks made on Experiment IX.; but lest the top rib should be overpowered, and by its compression the point of support be thrown lower down the beam, and consequently the beam weakened, the top rib was a little strengthened likewise.

---

* The castings in Experiments IX. and X. were broken at 4 feet distance between props, on account of defects near the end of the castings; the weight, however, was laid on the middle, 3 inches being taken off each end. The real breaking weights were 12068 and 9926 respectively; those given above being the reduced ones to a span of 4 feet 6 inches. From this cause the deflections are neglected.

The bottom rib will continue to be increased by small degrees, till such time as the beam breaks by compression, or by the separation of a wedge; at which point, perhaps, we shall have arrived at nearly the strongest form of section, for the same depth of beam and quantity of section.

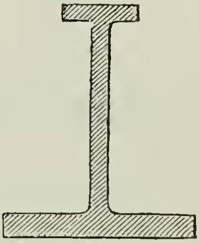
Experiment XI.

Beam from model of Experiment IX., only its top and bottom ribs altered as above.

Ratio of ribs 1 to 4 nearly.

Distance between supports and depth as before.

Fig. 16.



Dimensions of section.

Area of top rib = 1·6 × ·315 = 0·5 inches.
Area of bottom rib = 4·16 × ·53 = 2·2 „
Thickness of vertical parts . = ·38 „
Area of section . . . . . . = 4·50 „
Weight of beam . . . . . . = 57 lbs.

Deflection with 11186 lbs. ·4 inches.
„ 12698 „ ·45 „
„ 13706 „ ·52 „
Breaking weight = 14462 „ = 129 cwt. 14 lbs.

It broke by tension 1 inch from the middle; $b\,n = 2·5$ inches (see fig. 10).

Hence strength per inch of section $= \frac{14462}{4·5} = 3214$ lbs.

Comparing this with the result of Experiment XIII., which bore 2693 lbs. per inch—

$$3214 - 2693 = 521 = \text{excess.}$$

$$\therefore \text{Gain in strength} = \frac{521}{2693} = \tfrac{1}{5} \text{ nearly.}$$

We may seek for the gain by comparing the weights of the two beams, and the quantities they bore:* thus, in Experiment XIII., the weight of the beam was 41 lbs., and it broke with 8942 lbs.; and the weight of this beam is 57 lbs., and its breaking weight 14462 lbs.; hence 41 : 57 :: 8942 : 12431 = weight this beam should have borne, according to the strength of the common beam; but it actually bore 14462.

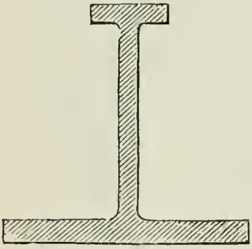
$$\therefore\ 14462 - 12431 = 2031 = \text{excess},$$

$$\text{and gain in strength } = \frac{2031}{12431} = \tfrac{1}{6} \text{ nearly.}$$

Experiment XII.

The model of this beam differed from the last in having a broader bottom flange.

Fig. 17.



Ratio of ribs 1 to $5\frac{1}{2}$ nearly.
Distance of supports as before.

Dimensions of section in inches.

Area of top rib $= 1{\cdot}56 \times {\cdot}315 = 0{\cdot}49$
Area of bottom rib $= 5{\cdot}17 \times {\cdot}56 = 2{\cdot}89$
Thickness of vertical part . . = ·34
Area of section . . . . . . = 5
Weight of beam . . . = $67\frac{1}{4}$ lbs.

---

* Let w = the weight of a beam of uniform dimensions, w' = the weight of a cubic foot of iron; then we readily find from Ex. I. p. 10,

$$\frac{\mathrm{W}}{w} = \frac{d}{l^2} \cdot \frac{\mathrm{C}}{144w'} = \frac{d\mathrm{C}'}{l^2},$$

$$\therefore \mathrm{C}' = \frac{\mathrm{W}}{w} \cdot \frac{l^2}{d}$$

where the value of C' determined by experiment for any particular form of beam enables us to ascertain its comparative strength.

If l and d are constant, then

$$\mathrm{C}' = \frac{\mathrm{W}'}{w}.$$

Similarly we have,

$$\mathrm{C}'_1 = \frac{\mathrm{W}_1}{w_1}$$

$$\therefore \frac{\mathrm{C}'}{\mathrm{C}'_1} = \frac{\mathrm{W}w_1}{\mathrm{W}_1 w}$$

which expresses the comparative strength of any two beams of the same length and depth.

Taking the above example, we have

$$\frac{\mathrm{C}'}{\mathrm{C}'_1} = \frac{14462 \times 41}{8942 \times 57} = 1\tfrac{1}{6} \text{ nearly,}$$

that is, the comparative gain of strength is $\frac{1}{6}$.

Weights in lbs.	Deflections.
8288	·24 inches.
12698	·36 ,,
13706	·40 ,,
14210	·42 ,,
15218	·45 ,,
15722	·48 ,,
16226	·49 ,,
16730	·53 ,,

With this last weight it broke, after having borne it some minutes. It broke by tension very near the middle, 16730 lbs. $=149$ cwt. 42 lbs.

Hence strength per square inch of section $=\frac{16730}{5}=3346$ lbs.

Comparing this with the result of Experiment XIII., we have $3346-2693=653=$ excess.

$\therefore$ gain in strength $=\frac{653}{2693}=\cdot 242=\frac{1}{4}$ nearly.

Seeking for the gain, by comparing the weight, $67\frac{1}{4}$, of this beam, and its breaking weight, 16730, with the weights, 41 and 8942, in Experiment XIII., we have, as in the last experiment, $41 : 8942 :: 67\frac{1}{4} : 14667$.

$\therefore$ $16730-14667=2063=$ excess, and gain in strength $=\frac{2063}{14667}=\frac{1}{7}$ nearly, which is considerably less than that given above, on account of the great weight of the bottom rib; it being uniform in size through its whole length of 5 feet.

Experiment XIII.

Beam of the *common form*, from the same model as the others.

Distance between supports as before.

Dimensions of section in inches (see fig. to Experiment IV.).

Thickness at A = ·29
,, B = ·425
,, C = ·53
,, DE = ·565
,, FE = 2·34
Area of section = 3·32 inches.
Weight of beam = 41 lbs.

Weights in lbs.	Deflections in parts of an inch.
7598	·4
8494	·43
8942	·47

With this weight it broke, after standing a few minutes. It broke $1\frac{1}{2}$ inches from the middle.

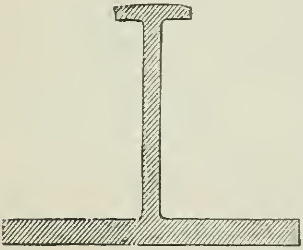
Hence strength per square inch of section $=\frac{8942}{3\cdot32}=2693$ lbs.

The beams in our future experiments were of equal height throughout their whole length (fig. 19), and had their top and bottom ribs uniform in thickness, but tapering in breadth towards the ends, the bottom rib being parabolic.

Experiment XIX.

Distance of supports, 4 feet 6 inches; depth of beam, $5\frac{1}{8}$ inches, as before.

Fig. 18.



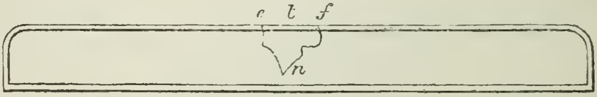
Dimensions of cross section in inches.

Area of top rib $=2\cdot33\times\cdot31=\cdot72$.
Area of bottom rib $=6\cdot67\times\cdot66=4\cdot4$.
Thickness of vertical part $=\cdot266$.
Area of section $=6\cdot4$, or $6\frac{2}{5}$ inches.
Weight of beam $=71$ lbs.

This beam broke in the middle by compression, with 26084 lbs., or 11 tons 13 cwt., a wedge separating from its upper side.

The weights were laid on gradually, and the beam bore them within a little of its breaking weight a considerable time, perhaps half an hour.

Fig. 19.



The form of the fracture and wedge is represented by fig. 19, showing a side view of the beam, where $e\,n\,f$ is the wedge, $e\,f=5\cdot1$ inches, $t\,n=3\cdot9$ inches, angle $e\,n\,f$ at vertex $=82°$.

It is extremely probable, from this fracture, that the neutral axis was at n, the vertex of the wedge, and therefore at three-fourths of the depth of the beam, since 3·9 inches $=\frac{3}{4}\times5\frac{1}{8}$ inches nearly.

Hence strength per square inch of section $=\frac{26084}{6\cdot4}=4075$ lbs., which is much greater than that in any of our former experiments.

Comparing this result with that of the common beam in Experiment XXII., which was cast with these, and which bore 2885 lbs. per inch, we have

$$4075-2885=1190 \text{ lbs.} = \text{excess.}$$

∴ gain in strength from the section $=\frac{1190}{2885}=\cdot41$, or upwards of two-fifths of what was borne by the common beam.

The quantity of metal saved, through the section, would be represented by the above excess, 1190, divided by 4075, the quantity which the beam bore per square inch of section.

∴ saving of metal from section $=\frac{1190}{4075}=\cdot292$, or $\frac{3}{10}$ nearly.

If we compare the strengths of this beam, and that in Experiment XXII., by the weights, we shall have the saving in metal, through the section and general form of the beam conjoined $=\cdot377$.

Thus we have, by constantly making small additions to the bottom rib, arrived at a point where resistance to compression could be no longer sustained; but it was not till the bottom rib had considerably more matter in it than double the rest of the beam, the bottom rib being to the rest as 4·4 to 1·83, and to the top rib as 6 to 1. Still the top rib was not crushed, nor did it exhibit any signs of weakness. The fracture took place by the vertical part of the beam becoming torn by the opposite forces of tension and compression round the neutral axis.

The great strength of this section is an indisputable refutation of that theory which would make the top and bottom ribs of a cast-iron beam equal.

Experiment XX.

Beam from the same model as that in the last experiment.
Distance between supports as before.

Dimensions of section in inches (see fig. last Experiment).

Area of top rib = 2·3 × ·28 = ·64.

Area of bottom rib = 6·63 × ·65 = 4·31.

Thickness of vertical part = ·335.

Area of section, 6·5, or $6\frac{1}{2}$ inches.

Weight of beam = $74\frac{3}{4}$ lbs.

Weights in lbs.	Deflections in parts of an inch.	Returned to (weights taken off).
9328	·22	·0
11397	·24	·0
12777	·25	·0
14345	·26	·03
15913	·30	·04
17481	·34	
18265	·36	
19049	·38	
20617	·43	
22185	·47	
22969	·48	
„	·50	

It broke in the middle of the beam by tension with 23249 lbs., or 10 tons 8 cwt. nearly.

This is considerably less than what the former beam bore, though its bottom rib, in which the tensile power of this form of section almost wholly lies, was not much different. The iron must therefore have been weaker.

Strength per square inch of section $= \frac{23249}{6\cdot5} = 3576$ lbs.

Comparing this with the result of the common beam in Experiment XXII., which bore 2885 lbs. per inch, we have 3576 − 2885 = 691 = excess.

$\therefore$ gain in strength from section $= \frac{691}{2855} = \cdot236$, in terms of what the common beam bore; whence the saving in metal $= \frac{691}{3576} = \frac{1}{5}$ nearly.

If we compare this beam with the common one, by their weights, the saving of metal will be ·26, or upwards of one-fourth.

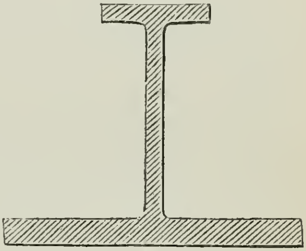
The thickness of the vertical part of the beam, in Experiment XIX., was ·266, and in this experiment ·335; we might therefore have increased the bottom rib of this beam in the ratio of 335 to 266, or by one-third nearly. With this distribution of the material it is probable that the beam would be upon the point of yielding to extension at the instant it was yielding to compression, or it might have yielded by the rupture of the vertical part, as in Experiment XIX. And thus a much greater excess of strength than that above found would have been obtained.

Experiment XXI.

This was on an *elliptical* beam, from the same model as that in Experiment XII., the bottom rib being further increased, and being, as in them, of equal breadth through the whole length of 5 feet.

Distance between supports as before.

Fig. 20.



Dimensions of section in inches.

Area of top rib = 1·54 × ·32 = ·493.
Area of bottom rib = 6·50 × ·51 = 3·315.
Thickness of vertical part = ·34.
Ratio of ribs, 6½ to 1.
Area of section = 5·41.
Weight of beam = 70¾ lbs.

Weights in lbs.	Deflections in parts of an inch.	Returned to (weight taken off).
9327	·26	·0
10707	·27	·0
11397	·28	·0
12087	·30	·0
12777	·31	·0
14345	·34	·0
15913	·35	·0
16697	·42	·06
17481	·43	·06
19049	·46	
19833	·50	
20617	·54	

It broke very near the middle, by tension, with 21009 lbs., or 9 tons 8 cwt. nearly.

Form of fracture nearly as *b n r* in figure; $bn = 1{\cdot}8$ inches.

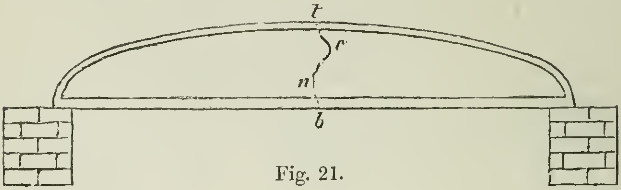


Fig. 21.

Hence strength per square inch of cross section $= \frac{21009}{5{\cdot}41} =$ 3883 lbs.

Comparing this with the result from the common beam in Experiment XXII., which bore 2885 lbs. per inch, we have $3883 - 2885 = 998 =$ excess.

Hence gain in strength $= \frac{998}{2885} = {\cdot}345$, in terms of what the common beam bore; or giving a saving in metal from section $= \frac{998}{3883} = {\cdot}257$, or upwards of one-fourth.

If the comparison be made by their weights, the saving in metal will be only ·23, which is less than it would have been had the ends of the beam been formed as in the preceding ones (XIX. and XX.), the bottom rib of this being all of one breadth and thickness, and 5 feet long, although the distance of the supports was only 4 feet 6 inches.

Experiment XXII.

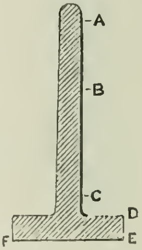
This beam was of the common form, from the same model as before, for comparison with the three preceding ones.

Distance between supports as before.

Dimensions of section in inches.

Thickness at A = ·30
" B = ·42
" C = ·45
" D E = ·51
" F E = 2·28
Area of section = 3·17
Weight of beam = 40 lbs.

Fig. 22.



This beam bore 8965 lbs., and broke in the middle with considerably less than 9327 lbs., a man supporting part of this extra weight by a lever. This accident prevented the exact determination, but I believe 9146 lbs., the mean between the numbers above, to be very near the breaking weight, perhaps rather above it.

Hence strength per square inch of section $= \frac{9146}{3 \cdot 17} = 2885$ lbs.

Rule for Strength of Cast-iron Beams.

Comparing the results of Experiments 9, 11, 12, 19, 20, and 21, and allowing for difference of iron, as indicated by the beams of the common form cast with the others for comparison, I find that the strength is nearly in proportion to the size of the bottom rib or flange; a bottom rib of double size giving nearly a double strength. And the subsequent experiments show the strength to be nearly as the depth, everything else being equal.* Therefore, in different beams, whose length is the same, the strength must be as their depths multiplied by the areas of a middle section of their bottom ribs; and where their lengths are different, the strengths will be as the product divided by the lengths,

$$\therefore \mathrm{W} = \frac{c\,a\,d}{l},$$

where W = the breaking weight in the middle of the beam, a= the area of a section of the bottom rib in the middle of the beam, d=the depth of the beam, l=the length or distance between the supports, and c=a quantity nearly constant (=26) in our best form of beams, and which will be supplied by taking the mean of the preceding experiments.

Example.—What weight laid on the middle of one of the main beams in the railroad bridge crossing Water Street, Manchester, would be required to break it, supposing it cast erect, and of the same iron as we have used in the experiments, the dimensions from the model now constructing by Messrs. Fairbairn and Lillie being as follow:—

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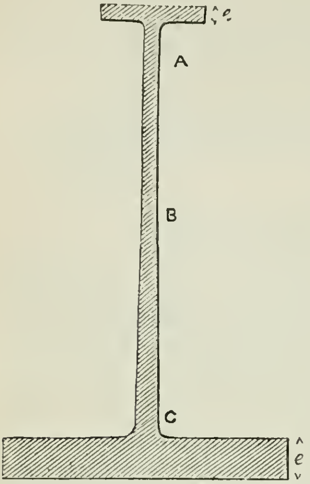
* At the same time it is important to bear in mind, that these formulæ are only strictly true in relation to similar beams. This theoretical deduction is fully confirmed by the results of the foregoing experiments.

Fig. 23.



Distance between supports, 26 feet, or 312 inches.

Depth of beam in middle, $27\frac{1}{2}$ inches.

Area of section of bottom rib in middle, $16 \times 3 = 48$ inches.

Form of section of beam nearly the same as annexed figure.

Referring to the formula we have

$$l = 312,\ d = 27{\cdot}5,\ a = 48$$

$\therefore$ W, the breaking weight,

$$= \frac{26 \times a \times d}{l} = \frac{26 \times 48 \times 27{\cdot}5}{312} = 110$$

tons.*

In the preceding experiments Mr. Hodgkinson, in arriving at the strongest section, compares nearly the whole of the results of his experiments with those obtained from the improved beam previously adopted by Mr. Lillie and myself. These comparisons, or the strength of each beam per square inch respectively, may, however, be accurately obtained from the following Table, which exhibits the weight, area of section, distance between the supports, breaking weights, &c., of the different beams which were in use for several years previous to the time at which Mr. Hodgkinson's experiments were made:—

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* The following formula, given by Mr. Tate in his treatise 'On the Strength of Materials,' is derived on the hypothesis that the areas of the top and bottom ribs are to each other in the inverse ratio of the force of compression to that of extension in the particular beam:—

$$\text{W} = \frac{16\, a_1\, (2d - e - e_1)^2}{l\, (2d + e_1 - e)} \text{tons.}$$

where a_1 = the area of the bottom rib, e_1 = the depth of the bottom rib, e = the depth of the top rib, d = the whole depth of the beam, l = the distance between the supports, and W = the breaking weight in tons.

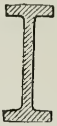
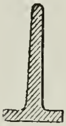
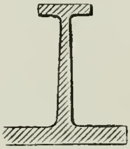
Taking the above example, we have, $a_1 = 48$, $e_1 = 3$, $e = 1$, $d = 27\frac{1}{2}$, $l = 312$; then

$$\text{W} = \frac{16 \times 48\ (2 \times 27\frac{1}{2} - 1 - 3)^2}{312\ (2 \times 27\frac{1}{2} + 3 - 1)} = 112 \text{ tons,}$$

which very nearly coincides with the result determined above.

This formula appears to be more strictly mathematical in its origin than the one which is commonly used, although they seem to agree very nearly in their results.

TABLE.

No. of Experiments.	Weight of beam in lbs.	Distance between supports.	Area of section in inches.	Deflection in inches.	Breaking weight in lbs.	Strength per square in. of section.	Remarks.
		ft. ins.					
1	36¼	4 6	2·82	—	6,678	2,368	 Tredgold's section of equal flanges top and bottom.
4	40½	4 6	3·20	·43	8,270	2,584	 Fairbairn's section of 1825 with single flange.
7	38	4 6	2·98	·63	9,503	3,188	
Mean.	39¼	4 6	3·13	·53	8,886	2,886	
19	71	4 6	6·4	·56*	26,084	4,075	 Hodgkinson's section of greatest strength, with areas of flanges as 6 to 1.
20	74¾	4 6	6·5	·50	23,249	3,576	
Mean.	72·82	4 6	6·45	·53	24,666	3,825	

Adopting Mr. Hodgkinson's calculations, we then have the value of the different sections of beams experimented upon by Tredgold, by myself, and by Mr. Hodgkinson, as the numbers 236, 288, and 382; or, taking Mr. Hodgkinson's section of greatest strength as unity, the ratios will stand—

For Hodgkinson and Fairbairn as . 1 : ·754
For Hodgkinson and Tredgold as . 1 : ·619
And for Fairbairn and Tredgold as . 8 : ·820

These numbers appear to give the relative strengths of the different beams, and they no doubt were the strongest sections of all beams in use at the respective dates. It is to be regretted that we are not in possession of any comparative experiments on the original beam of Boulton and Watt's section. By a simple calculation, however, we find the ratio of strength to be as 1 : ·543; that is to say, the resisting power of one of these beams was little more than one-half that of Mr. Hodgkinson's strongest beam. The increased demand for fire-proof buildings, taken in connection with the attainment of the strongest section for cast-iron beams, gave a renewed impulse to their application in every direction. The old form of beams introduced by myself and Mr. Lillie with the single flange, and those by Mr. Tredgold with equal flanges, were discarded in order to make way for those of the improved section; and the confidence of engineers in the security of iron beams was so much strengthened that the span, or the distance between the supporting columns of fire-proof buildings, was increased from 14 to 20 feet. This power of enlargement occurred most opportunely, as the amplification, or rather the longitudinal extension, of some of the principal machinery in cotton mills, at that particular period, necessitated a considerable increase in the width of the mills. Such, moreover, was the confidence inspired by this improved section, that I have myself constructed buildings from 6 to 7 stories in height, and 52 feet wide, with only one row of pillars down the centre of each room, and two beams across, each 26 feet span between the centre columns and the walls on each side.

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* The deflection, ·56 inches, is computed from Experiment XX.; the deflections not being given in Experiment XIX. The ultimate deflection of Experiment No. 1 on Mr. Tredgold's beam of greatest strength is necessarily omitted, and is not recorded in Mr. Hodgkinson's paper.

In the construction of the floors of warehouses which have to support heavy weights,* this section of beam (when made sufficiently strong) has been found perfectly secure; and in bridges also, where the span does not exceed 40 feet, it may be used with perfect safety if proper precautions are taken to insure sound and perfect castings.

On one occasion, and I believe only one, a girder bridge has been erected with beams 76 feet span all in one casting. They were made in this country for Messrs. John Dixon & Co. of Amsterdam, and were erected by those gentlemen on some part of the Haarlem Railway.

Notwithstanding the increased security which has been gained by these improvements in the form of cast-iron beams, their use is nevertheless attended with danger when either the design or construction are confided to the hands of ignorant persons; and the numerous and fatal accidents which have occurred at various times, and which have very naturally created in the public mind serious apprehension as to their security, have almost invariably been traced to this cause. On more than one occasion, as many as from fifteen to twenty lives have been lost by the failure of cast-iron beams in factories and buildings where numbers of people were congregated; and the aggregate loss of life and property from this cause has been very serious. One of the most alarming accidents of this kind happened at Oldham, on the 31st of October, 1844, in consequence of the breaking of one of the beams of a cotton factory. In this case upwards of twenty persons were buried in the ruins.*

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* See Part III., on the Construction of Fire-proof Buildings.

Among many other catastrophes of the same nature may also be instanced that at Mr. Nathan Gough's mill at Salford in 1828, where the beams were of a similar construction to those used in the building of the jail at Northfleet, and the more recent one at Mr. Gray's mill in Manchester in 1845, a description of which was given the same year to the Institution of Civil Engineers.

ON COMPOUND OR TRUSSED CAST-IRON BEAMS OR GIRDERS.

In the Government report just referred to, several important facts were recorded, bearing directly upon the dangerous nature of trussed girders, or that description of girders where attempts are made to increase their strength, and to maintain them in form, by the use of wrought iron bars fastened at the upper ends, and acting in a diagonal direction on the bottom of the beam. Of the safety of these tension-rods I have always had serious apprehensions; but as many other persons of highly distinguished attainments hold a different opinion, it may not be considered irrelevant if I adduce my reasons for the view which I take, and the experiments upon which those reasons are founded.

If we take a cast-iron beam of the section of greatest strength, and endeavour, by means of truss-rods, similar to A B C in the following sketch, to increase its powers of resistance, we shall find that, under certain circumstances, they introduce an antagonistic force, which has an injurious influence; or that, in other words, the beam would be safer without the truss-rods than with them.

To some this may appear paradoxical; but in order to ascertain how far the statement is entitled to credit, let us assume the flanges *a*, *a* (fig. 24), to be one-sixth of the area of the flange *b*, *b*;* and under the impression of still further adding to its strength, let us suppose that two truss-rods, A B, B C (fig. 24),

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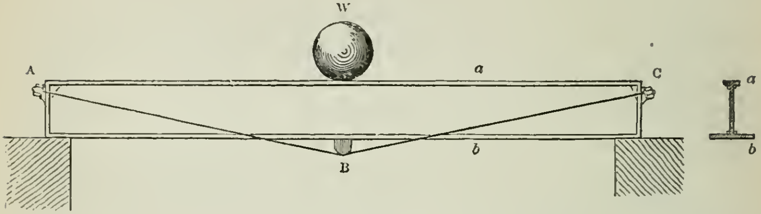
* See Report to the Government Commission, in Appendix No. II.

Fig. 24.



are applied, one on each side of the beam, to assist in supporting the weight W.

Experimentalists having found that wrought iron possesses great powers of resistance to extension, while cast iron presents great powers of resistance to compression, it became a matter of inquiry how far, and under what circumstances, cast iron and wrought iron might be employed together in the construction of beams, so as to embrace the advantages arising from these peculiar properties of the two materials. This inquiry gave rise to the construction of truss-beams, where the wrought iron is solely employed to give strength to the bottom part of the beams by its tensile resistance, while the cast iron in the top part of the beams is solely employed to resist the force of compression. Now, if a truss-beam could be constructed so that the two materials might be brought to act in perfect concert with each other, this contrivance would, no doubt, effect a considerable economy of material; but we shall hereafter show that this is impracticable.

In a perfect truss-beam (supposing it possible to have such a thing) the cast metal should be upon the point of rupture by compression at the same moment that the truss-rods are about to yield to extension. If too great a tension is given to the rods, they will break before the beam has arrived at the condition of rupture; on the contrary, if too small a tension is given to them, the beam will break before they have arrived at their condition of rupture. In the absence of exact data, we should say, in order to avoid danger, that the tension of the truss-rods had better be too low than too high; for in the former case they would yield up a portion of their tensile resistance, and then leave the remaining portion of the load to be borne by the beam itself. Experiment I., p. 41, shows the difficulty of adjusting the tension of the truss-rods; for in this case they yielded to extension, and then the beam broke with a weight which it would have nearly sustained by its own resisting powers. In order to discover the best tension for the truss-rods, it is necessary that we should consider more minutely the distinctive properties of the two metals composing the truss-beam.

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* These proportions, as we have shown by experiment, constitute the strongest sectional form.

The two kinds of material are very different in their physical as well as in their mechanical properties. *Cast iron* is a hard, rigid, crystalline, unmalleable substance, which presents a great resistance to a force of compression, but a comparatively small resistance to that of extension; and from its low degree of ductility, it undergoes but little elongation when acted upon by a tensile force. On the contrary, *wrought iron* is a flexible, malleable, ductile substance, which presents a great resistance to a force of extension, but a somewhat less resistance to that of compression: from its high degree of ductility, it undergoes a considerable elongation when acted upon by a tensile force. When the two metals are released from the action of a tensile force, the SET of the one metal differs widely from the SET of the other. The flexibility of wrought iron is from eight to ten times greater than that of cast iron. Under the same increase of temperature the expansion of wrought iron is considerably greater than that of cast iron. While wrought iron yields to a stroke, cast iron is readily broken by a severe collision, or by any violent vibratory action.

The following generalisations of an extensive series of experiments will give an exact comparative view of these properties of cast iron and wrought iron.

TABLE I.—MEAN ELONGATIONS BY TENSILE FORCES WITHIN THE LIMITS REQUISITE TO RUPTURE CAST IRON, VIZ. ABOUT $7\frac{1}{2}$ TONS PER SQUARE INCH OF THE TRANSVERSE SECTION.

Name of the Metal.	Mean elongation, the force being 1 ton per square inch.	Ratio of elongations.	Sets with 7 tons per square inch.
Cast iron . .	$\frac{1}{5450}$ part of the whole length of the bar .	$2\frac{1}{4} : 1$	$\frac{1}{7}$ of the whole elongation.
Wrought iron .	$\frac{1}{12300}$ part of the whole length of the bar .		$\frac{1}{112}$ of the whole elongation.

From this Table it appears, that for forces of extension below $7\frac{1}{2}$ tons per square inch, the mean elongation of cast iron is about $2\frac{1}{4}$ times that of wrought iron. When the cast iron is about to undergo rupture, its ultimate extension is about 3 times that of the wrought iron. Moreover, the set of the cast iron, within this limit, is considerably greater than that of the wrought iron.

Table II.—Mean elongations and sets, with tensile forces equal to two-thirds of the forces requisite to produce rupture in each case respectively.

Name of the Metal.	Force per square inch, in tons.	Elongation on 10 feet of the bar, in inches.	Ratio of elongations.	Set.	Ratio of sets.	Proportion of sets to elongation.
Cast iron .	5	·114	$1 : 2\frac{2}{5}$	·013	1 : 10	$\frac{1}{9}$
Wrought iron .	15	·275		·133		$\frac{1}{2}$

From this Table it appears, that when the parts of the truss-beam are duly loaded, the conditions are reversed; that is to say, the elongation of the wrought iron becomes $2\frac{2}{5}$ times that of the cast iron, and the set of the former becomes ten times that of the latter.

Table III.—Mean values of the tensile forces requisite for producing equal elongations in cast-iron and wrought-iron bars 10 feet long, with the corresponding sets.

Mean elongations on 10 feet, in inches.	Cast iron. Force per sq. in. in tons.	Wrought iron. Force per sq. in. in tons.	Cast iron. Set in inches.	Wrought iron. Set in inches.
·005	·26	·56		
·024	1·11	2·5	·0012	Not perceptible
·04	2	4·5	·0031	·0005
·05	2·5	5·6	·0043	·0007
·062	3	6·76	·0056	·0009
·087	4	9	·009	·0027
·129	5·5	12·4	·0159	·014
·145	5·9	13·26	·019	·043

This Table establishes the following remarkable law relative to the forces requisite for producing equal elongations in cast-iron and wrought-iron bars: Within the limit of 6 tons tensile strain per square inch for cast iron, and $13\frac{1}{2}$ tons for

WROUGHT IRON, THE TENSILE FORCE APPLIED TO WROUGHT IRON MUST BE 2¼ TIMES THE TENSILE FORCE APPLIED TO CAST IRON, IN ORDER TO PRODUCE EQUAL ELONGATIONS.

This result is consistent with that of Table I., where the elongation of cast iron, for equal increments of force, is shown to be 2¼ times that of wrought iron. The elongations in this Table may be approximately derived from Table I.

Further, with a force of about 5½ tons applied to cast iron, and 12½ tons to wrought iron, the sets, as well as the elongations, are nearly equal to each other. Now, if these forces had been duly apportioned to each other, this circumstance would have given us an eligible principle for adjusting the tension of the iron rods in a truss-beam; but unfortunately this strain upon the cast iron is too near the strain requisite for producing rupture, while that upon the wrought iron is only about one-half its greatest tensile resistance. For forces below 5½ and 12½ tons, the set of the cast iron is greater than that of the wrought iron; and for forces above 5½ and 12½ tons, the reverse takes place.

TABLE IV.—ULTIMATE ELONGATIONS, THE CAST IRON BEING LOADED WITH 7½ TONS PER SQUARE INCH, AND WROUGHT IRON WITH 24 TONS PER SQUARE INCH.

Name of the Metal.	Total ultimate elongation, in parts, of the length of the bar.	Ultimate elongation per ton, in parts, of the length of the bar.
Cast iron . .	$\frac{1}{550}$, or ·22 in. on 10 ft. .	$\frac{1}{4000}$
Wrought iron .	$\frac{1}{21}$, or 5·7 in. „ . .	$\frac{1}{520}$

Hence it follows, that the ultimate elongation of wrought iron per ton on each square inch is about 8 times that of cast iron, and that the total ultimate elongation of wrought iron is about 26 times that of cast iron.

If we take the results of Mr. Loyd's experiments,* where the average of the breaking weights was 32 tons per square inch, we shall find that the total ultimate elongation of wrought iron is about 130 times that of cast iron.

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* See the author's *Experimental Inquiry into the Strength of Wrought-iron Plates, &c.*, published in the Transactions of the Royal Society for 1850.

Table V.—Permanent set of bars, expressed in parts of their elongation.

Weights in tons per square inch.	Cast iron. Set in parts of the elongation.	Wrought iron. Set in parts of the elongation.
2	$\frac{1}{13}$	Scarcely perceptible.
3	$\frac{1}{11}$	
5	$\frac{1}{9}$	$\frac{1}{110}$
7	$\frac{1}{7}$	$\frac{1}{112}$
10	. . . .	$\frac{1}{60}$
15	. . . .	$\frac{1}{2}$
20	. . . .	$\frac{11}{12}$

Here it will be seen, that for weights below $7\frac{1}{2}$ tons the set of cast iron is incomparably greater than that of wrought iron; on the contrary, for weights above 15 tons, the set of wrought iron is considerably greater than the maximum set of cast iron.

Table VI.—Mean elongation of cast-iron and wrought-iron bars 10 feet long, by an increase of 90° temperature.

Length of bar 10 feet.	Elongation due to 90° increase of heat.	Difference of the elongations on 10 feet.
Cast iron . . .	·0666 inches	·0067 inches.
Wrought iron . .	·0733 „	

Comparing the results of this Table with those of Table I., we find that the elongation of wrought iron by an increase of 90° temperature is equivalent to the action of a tensile force of 7·4 tons per square inch; and that of cast iron to a force of 3 tons per square inch. Moreover, the difference of the elongations of the two metals is equivalent to the action of a tensile force of $\frac{3}{4}$ tons per square inch. It is also worthy of remark, that while making experiments relative to the elongations of metals when acted upon by tensile forces, we should carefully observe that the temperature remains nearly the same.

From a careful induction of the facts contained in these Tables, let us endeavour to determine the best adjustment of the tension of the truss-rods.

First. Let us consider the case, when the truss-rods have no strain upon them at the time the beam is unloaded.

Suppose the beam to be loaded so as to produce a tensile strain upon the cast iron equal to one-third its breaking-load, that is to say, let the force of elongation be $2\frac{1}{2}$ tons per square inch upon the cast metal; then, from Table III., we find that the strain upon the truss-rods will be about $5\frac{1}{2}$ tons per square inch, and that the set of the cast iron, after these strains are taken off, will be 6 times that of the wrought. Now, in this case, while the cast iron is strained to one-third its breaking weight, the wrought iron is strained to only about one-fifth its ultimate strength; and, further, when the load is taken off, the cast-iron beam will remain much more elongated than the iron rods, which will, to a certain extent, destroy their original adjustment of tension, but this, in the present case, will not act unfavourably, for it will tend to give a certain amount of tension to the truss-rods.*

Suppose the beam to be loaded so as to produce a tensile strain upon the cast iron equal to $5\frac{1}{2}$ tons per square inch; then in order to produce an equal elongation of the truss-rods, the strain upon them must be $2\frac{1}{4}$ times $5\frac{1}{2}$ tons, or $12\frac{1}{2}$ tons nearly. Here, while the cast iron is strained to more than two-thirds its ultimate resistance, the wrought iron is only strained to about one-half its ultimate resistance. One favourable circumstance connected with this load is, that the sets of the two metals are very nearly the same.

Suppose the beam to be loaded so as to produce a tensile strain of 15 tons per square inch upon the truss-rods, then, by Table II., this will produce an elongation of $\frac{1}{440}$ part of the length of the rod; but, by Table IV., the ultimate elongation of cast iron is $\frac{1}{550}$ part of its length; therefore the cast iron would be ruptured by extension some time before the truss-rods could arrive at a strain of 15 tons per square inch, that is, before they could be strained to two-thirds their ultimate strength.

This adjustment is defective: the truss-rods must obviously have a certain amount of tension, before the load is laid on, in order to bring them into a higher condition of action, and to counteract the set of the cast metal.

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* We have here considered the length of each truss-rod to be one-half the length of the beam. This supposition will obviously involve no appreciable amount of error.

Second. Suppose the truss-rods to be screwed up so as to give them a tension of 8 tons per square inch, or one-third their breaking tension; and, for the sake of simplicity, let us suppose that the half-length of the beam is 10 feet. This high tension of the truss-rods, it should be observed, will produce a dangerous action upon the cast metal.

Suppose the beam to be loaded so as to produce a tensile strain of $7\frac{1}{2}$ tons per square inch upon the cast metal. Now, by Table IV., this would give an elongation of ·22 inches; but the truss had an elongation of ·077 inches due to the strain of 8 tons when the beam was in a neutral condition; therefore the total elongation of the truss-rod will be $\cdot 22 + \cdot 077$, or ·297 inches; but from Table II. we find this elongation to correspond to about 16 tons per square inch tensile force upon the rods. Thus, it appears, that even with the dangerous tension of 8 tons per square inch on the truss-rods, we cannot produce a higher strain than 16 tons upon them at the moment when the cast iron is about to rupture.

Reasoning in this manner, it may be shown, that it is impossible to construct a truss-beam which shall task the high tensile resistance of wrought iron without at the same time introducing a dangerous action upon the cast metal. We have shown, in Tables II. and IV., that for high proportional tensions, the rate of elongation of wrought iron is from 10 to 26 times that of cast iron; hence it is impossible to have the two metals acting in concert at tensions approaching their rupture.

Since little is gained by this high tension in point of ultimate strength, and much is lost by the injury done to the beam, we must reduce this tension in order to arrive at the best form of the truss-beam.

Third. Let us endeavour to discover the tension which must be given to the truss-rods, so that the different parts of the truss-beam may be respectively loaded, at the same moment, with one-third their respective ultimate tensile resistances, viz. $2\frac{1}{2}$ tons per square inch for the cast iron, and 8 tons per square inch for the wrought iron.

Here, by the law of Table III., the additional force tending to elongate the iron rods per square inch $= 2\frac{1}{4} \times 2\frac{1}{2} = 5\frac{5}{8}$ tons.

Putting t = the tension of the rods per square inch at the moment when the cast metal has no strain upon it, we have

$$t+5\tfrac{5}{8}=8,$$
$$\therefore\ t=2\tfrac{3}{8} \text{ tons per square inch, or } 2\tfrac{1}{2} \text{ tons nearly.}$$

Suppose the beam to be loaded so as to produce a tensile strain of 4 tons per square inch of the cast metal, then the truss-rods will undergo an additional strain of $2\frac{1}{4}$ times 4 tons, or 9 tons per square inch, which, added to $2\frac{1}{2}$ tons, will give $11\frac{1}{2}$ tons for the whole strain per square inch of the truss-rods; so that the two materials will be loaded to about one-half their respective breaking weights; and, moreover, it may be shown from Table III., that the sets of the two metals after the load is taken off will be nearly the same.

Hence it appears that the most eligible adjustment of the truss-rods is to give them a tension of from 2 to 3 tons per square inch.

But a load of $5\frac{1}{2}$ tons per square inch on the cast metal would tend to destroy the adjustment; for this would produce a strain of about $13\frac{1}{2}$ tons per square inch on the truss-rods; and after the load is taken off, the set of the wrought iron would be about 3 times that of the cast metal. It may be further observed, that a strain of less than 15 tons per square inch upon the wrought iron would rupture the cast metal.

An ordinary beam (especially when the material is wrought iron) may be safely loaded, to meet contingencies or particular exigencies, within two-thirds of its breaking load; but this cannot be done with truss-beams; for, with the best adjustment of the trusses, as we have shown, the cast metal will be upon the point of rupture before the wrought iron has attained two-thirds its ultimate resistance.

Upon the whole, it appears to be impracticable to attain such an adjustment of the parts of a truss-beam as to secure the safety of the beam, with a due regard to the most efficient action of all its parts. The two materials are so different in their physical as well as in their mechanical properties, that it seems impossible to construct a beam with them where they can, under all ordinary strains, act in concert with each other. But, even supposing that we are able to construct a truss-beam with all its parts perfectly adjusted, how long would it remain so? Besides the disturbances arising from unequal elongations and sets; sudden collisions, changes of temperature, and other causes, would tend to destroy this adjustment. The defect of a truss-beam consists, not so much perhaps in its want of economy as regards the distribution of material, as in its want of stability and safety;—within comparatively small limits of load, a truss-beam may pass from a condition of perfect security and safety to one of uncertainty and danger.

Approximate Rule for calculating the strength of a Truss-Beam.

In order to calculate the strength of trussed beams, let us suppose that the tension of the rods is such as to cause them to have a strain of 8 tons per square inch, at the same moment that the cast iron has a strain of $2\frac{1}{2}$ tons per square inch; then with this *perfect adjustment*, we have found, by a process of reasoning which need not be given in this place, the following approximate rule for calculating the weight with which the beam may be safely loaded:—

Add three times the section of the truss-rods to the section of the bottom flange, substitute this sum for the bottom area in the usual formula for calculating the strength of cast-iron beams, and one-third this result will give the weight of safety, or one-third the theoretical breaking weight.

Thus, let w = the load of safety, a = the area of the bottom flange, a_1 = the section of the truss-rods, l = the distance between the points of support, and d = the depth of the cast-metal beam; then

$$w = \frac{26\,(a + 3a_1)\,d}{3l} \text{ tons} \quad . \quad . \quad (1).$$

In the first series of experiments, given at p. 40, we find, $a = 1{\cdot}05$, $a_1 = {\cdot}39$, $d = 4$, $l = 4{\cdot}5 \times 12$,

$$\therefore w = \frac{26\,(1{\cdot}05 + 3 \times {\cdot}39)\,4}{3 \times 4{\cdot}5 \times 12} = 1{\cdot}4 \text{ tons} = 3{,}100 \text{ lbs. nearly.}$$

Now the breaking load of this beam, as given in Experiment III., Table I., p. 40, was nearly 9,000 lbs., giving one-third of 9,000 lbs. = 3,000 lbs., for the load of safety. Hence it appears that in this truss-beam we had very nearly hit upon a perfect adjustment.

Throughout these calculations, we have assumed that the section of the top flange of the beam is duly calculated to balance the united tensile forces of the truss-rods and the bottom flange of the beam.

Let us now consider the question of economy, as regards these beams.

Comparison of Cost.

In estimating the comparative advantages of different forms of beams, we should always consider their ratio of cost for a given amount of strength. In order to apply this method of comparison to the case of trussed beams, let a = the cost of the beam without the trusses a_1 = the cost of the truss-rods a_2 = the cost of their construction, w = the breaking weight of the beam without the trusses, and w the breaking weight with the trusses; then we have—

Comparative advantage of the trussed beam, that of the beam without the trusses being unity

$$=\frac{a}{a+a_1+a_2}\times\frac{\mathrm{W}}{w}\quad\ldots\ (1).$$

In the case of the beam experimented upon, see Table I., p. 40, we have—

$a=4\frac{1}{2}$ shillings, $a_1+a_2=4$ shillings, $w=5800$, $\mathrm{w}=7400$; then by formula (1) we have—

Comparative advantage of the trussed beam

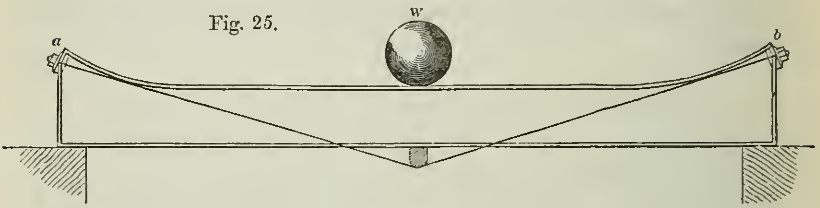
$$=\frac{4\frac{1}{2}}{4\frac{1}{2}+4}\times\frac{7400}{5800}=\tfrac{2}{3}\text{ nearly};$$

that is to say, the advantage of the simple beam, as compared with the trussed beam, is nearly as 1 to $\frac{2}{3}$.

Trussed Beams where the Rods are attached to points above the line of the top flange.

Let us now consider that form of a truss-beam, where the upper ends of the rods are attached to points, *a* and *b*, lying above the line of the top flange of the beam, as represented in the following figure :—

Fig. 25.



This form of construction, in my opinion, rather tends to increase than to diminish the defects of the truss-beam ; for the raising of the points of attachment, *a* and *b*, increases the power of the truss-rods to rupture the upper flange of the beam. And we may here observe, that this objection applies with equal force to a compound beam of this form, where the whole of the material is wrought iron.

Reverting to the beam of the best form and strongest section, it will be seen (Table, p. 43), that on turning it upside down, with the broad flange uppermost, we then have in its simple form a comparatively weak construction, which breaks with 3,366 lbs. instead of 5,830 lbs., the measure of strength when the small flange intended to resist compression is uppermost. These facts are corroborated by direct experiment. Hence it follows, that there is a loss of nearly one-half the strength by the change of position alone.

Now, if we endeavour to remedy this defect by the application of an auxiliary power, in the shape of a truss, to the weak side of the beam, we then bring into operation entirely new elements of strength ; and provided the truss-rods are strong enough, we may reasonably calculate on having a more powerful structure than the simple beam, in its right position, without auxiliary support. In fact, the resistance to compression is increased six times; and assuming that the truss-rods are equally secure in the direction of tension, we should then have a beam capable, theoretically, of supporting the enormous pressure of nearly six times the breaking weight.

This power of resistance, however, is computed on the assumption that the two resisting forces are equal, and that all the other parts of the beam are so constructed, that they will maintain the connection of the different parts, and prevent disruption either in a lateral or in a vertical direction.

We have admitted that the truss-rods may be useful in the position represented by fig. 25, but inasmuch as the casting would require to be altered in form, and strengthened in other parts, it becomes questionable whether one equally strong could not be made of homogeneous construction, of one casting, that would carry the same weight. This must, however, be left to the judgment of the architect or engineer, as much depends upon the size and weight of the beam and the width of the span, which should never exceed 30 feet where homogeneous or cast-iron beams are employed. In cases where the span exceeds that distance it is safer to employ wrought-iron beams, as their construction is always to be depended upon, being free from flaws and other imperfections to which cast iron is subject. It is true that beams have been cast and are now in use upwards of 40 feet long; but this is no argument in favour of such large castings, which are always precarious and untrustworthy, in cases of wide span, where a superior and much safer material *can be employed at the same cost.*

Considering the difference of opinion which exists on this subject, I have deemed it necessary to examine the question experimentally; and in order to arrive at correct conclusions, model beams were prepared and submitted to the usual tests, with the following results:—

Experiments to determine what advantages, if any, are derived from Wrought-iron Trusses applied to Cast-iron Beams as an additional support:—

EXPERIMENTS ON A BEAM OF THE STRONGEST SECTION, 4 FEET 6 INCHES BETWEEN THE SUPPORTS AND TWO TRUSS-RODS, EACH ONE HALF-INCH DIAMETER TO THE BOTTOM AND ENDS, WITH THE BROAD FLANGE BELOW.

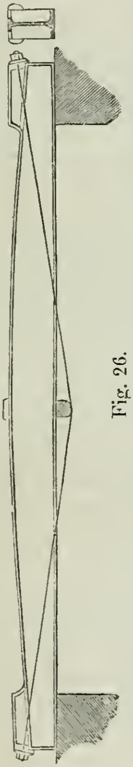


Fig. 26.

Experiment I.			Experiment II.			Experiment III.		
No. of Experiment.	Weight laid on in lbs.	Deflection in inches.	No. of Experiment.	Weight laid on in lbs.	Deflection in inches.	No. of Experiment.	Weight laid on in lbs.	Deflection in inches.
1	406	·05	1	406	·045	1	406	·012
2	910	·10	2	874	·076	2	1,574	·120
3	1,358	·15	3	1,322	·130	3	2,470	·152
4	1,806	·20	4	1,770	·173	4	3,366	·223
5	2,254	·23	5	2,462	·234	5	4,262	·284
6	2,478	·28	6	2,910	·287	6	5,158	·326
7	2,702	·31	7	3,358	·342	7	5,606	·400
8	2,926	·34	8	3,806	·404	8	6,056	·470
9	3,150	·37	9	4,354	·440	9	6,502	·528
10	3,262	·38	10	4,578	·485	10	6,950	·571
11	3,374	·40	11	4,690	·500	11	7,398	·620
12	3,486	·41	12	4,802	·531	12	7,846	·670
13	3,710	·44	13	4,914	·560	13	8,294	·735
14	3,934	·47	14	5,026	·584	14	8,742	·780
15	4,046	·48	15	5,138	·600	15	8,854	Broke
16	4,158	·50	16	5,250	·620			
17	4,290	·51	17	5,304	·640			
18	4,382	·52	18	5,496	·672			
19	4,494	·53	19	5,610	·690			
20	4,606	·55	20	5,722	·710			
21	4,718	·56	21	5,834	·726			
22	4,830	·57	22	5,946	·740			
23	4,942	·59	23	6,058	·760			
24	5,054	·61	24	6,160	·772			
25	5,166	·62	25	6,272				
26	5,278	·64	26	6,384				
27	5,390	·65	27	6,496				
28	5,502	Broke	28	6,608	·826			
			29	6,720				
			30	6,832				
			31	6,934				
			32	7,046				
			33	7,136				
			34	7,270				
			35	7,384				
			36	7,496				
			37	7,608	·934			
			38	7,720				
			39	7,832				
			40	7,944	Broke			

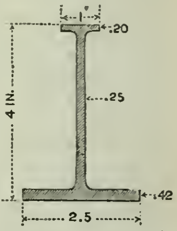
from one of the truss-rods yielding to tension.

Ultimate deflection } ·66

Ultimate deflection } ·988

Ultimate deflection } ·790

Fig. 27.



Area of bottom flange } 1·05

Area of top flange } ·20

Bottom flange, 2·5 × ·42, area = 1.05 inches.

Top flange 1·0 × ·20, area = ·20 inches.

Depth of beam in the middle, 4 inches.

EXPERIMENT IV.

THE SAME BEAM, WITH THE BROAD FLANGE BELOW, AND WITHOUT TRUSS-RODS, 4 FEET 6 INCHES BETWEEN SUPPORTS.

No. of Experiment.	Weight laid on in lbs.	Deflection in inches.
1	406	·70
2	1,574	1·50
3	2,022	2·86
4	2,470	3·26
5	2,918	4·20
6	3,366	4·91
7	3,814	5·60
8	4,262	6·31
9	4,710	6·82
10	5,158	7·25
11	5,382	7·50
12	5,606	7·76
13	5,830	Broke
	Ultimate deflection	7·86

Fig. 28.

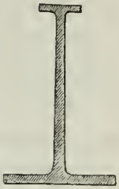
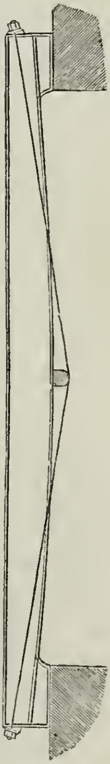


Fig. 29.



EXPERIMENT V.

THE SAME BEAM, WITH THE BROAD FLANGE ABOVE AND TWO TRUSS-RODS, THREE-QUARTERS OF AN INCH DIAMETER, 4 FEET 6 INCHES BETWEEN SUPPORTS.

No. of Experiment.	Weight laid on in lbs.	Deflection in inches.
1	406	·03
2	1,574	·07
3	2,470	·10
4	3,366	·16
5	4,262	·20
6	5,158	·24
7	6,054	·30
8	6,950	·36
9	7,846	·47
10	8,742	·53
11	9,638	·61
12	10,086	·69
13	10,534	·77
14	10,972	·84
15	11,420	·91
16	11,868	1·02
17	12,316	Broke
	Ultimate deflection	1·06

In the first experiment one of the truss-rods broke after sustaining the weight, 5,502 lbs., some seconds. The second and third broke, through the upper flange, by compression. Taking, however, the mean of the whole, we have the breaking weight at 7,433 lbs.

Owing to the failure of one of the truss-rods, the experiment was twice repeated with $\frac{3}{4}$-inch rods, in order to produce fracture in the cast iron, before the rods could yield to tension.

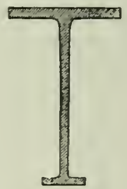
The beam in this position would have carried a much greater load if the truss-rods had been stiffer, as the lower part of the beam was torn asunder from the increased deflection, before the top flange had arrived at its ultimate powers of resistance to compression.

Experiment VI.

The same Beam reversed, with the Broad Flange uppermost and without Truss-rods, 4 feet 6 inches between Supports.

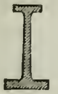
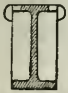
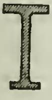
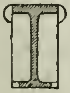
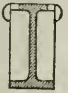
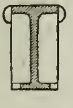
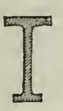
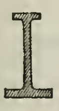
No. of Experiment.	Weight laid on in lbs.	Deflection in inches.
1	406	·080
2	1,574	·132
3	2,022	·226
4	2,470	·362
5	2,918	·473
6	3,142	·514
7	3,366	Broke.
	Ultimate deflection	·55

Fig. 30.



The experiments above recorded present for consideration the strengths of beams, with the following varieties of form and disposition. 1st, The cast-iron beam, in its best form, with the broad flange below, when assisted by a wrought-iron truss. 2d, The same beam reversed, with the broad flange above, and supported by a truss. 3d, The same beam, broad flange above, and without a truss. And, lastly, the same beam, in its strongest form and position, also without a truss. Now, all these conditions admit of comparison. In working them out in detail, we have the following results and ratios of strengths:—

SUMMARY OF RESULTS.

Sketch of Beam.	Description of Beam.	Breaking weight in lbs.	Ratio of strength in lbs.
	Cast-iron beam, with the broad flange downwards . . .	5,830	100 : 127
	The same beam in the same position, with double truss-rods supporting the middle	7,433	
	Beam reversed; the broad flange uppermost without truss-rods	3,366	100 : 365
	The same in the same position, with broad flange supported by truss-rods . .	12,316	
	Beam with double truss, as before; broad flange below.	7,433	100 : 165
	The same, with double truss; broad flange uppermost .	12,316	
	Beam without truss-rods; broad flange uppermost .	3,366	100 : 173
	The same beam, broad flange downwards	5,830	

From the above summary we may draw the following conclusions:—

1st, That the advantage gained by adding truss-rods to a cast-iron beam of the strongest section, and placed in the best position for resisting a transverse strain, is as 100 : 127, being rather more than one-fourth of increase in strength.*

2nd, That the simple beam reversed, with the small flange downwards, loses nearly one-half of its strength, as compared with the same beam in its most favourable position, with the large flange downwards, or, as 100 : ·62. Again, let the beam be reversed and trussed, with the small flange downwards, and its resisting powers are increased $3\frac{1}{2}$ times in strength as compared with the same beam in the same position without the truss, or, as 100 : 365.

Lastly, That the same beam, being trussed in both instances, first, with the broad flange downwards, and subsequently with the broad flange upwards, gains nearly three-fourths in strength in the latter case; whilst the other is not materially increased beyond that of the simple beam entirely free from auxiliary support.

We might multiply these comparisons to a much greater extent; but we have done sufficient to prove the fact, that under the most favourable circumstances, there is not much gained in the strength of cast-iron beams by the addition of malleable-iron truss-rods, whilst their uncertain and variable character render them dangerous in actual use. When such auxiliaries become absolutely necessary, I would then recommend them to be attached to beams with a strong flange on the upper side to resist compression, and the tension-rods so regulated and proportioned in strength as to cause them to act simultaneously with the rigid top in their resistance to fracture. What is, however, infinitely preferable, is a well-constructed malleable-iron beam, which may be made of almost any given strength, and of any span within the limits of 500 to 1000 feet.

General Remarks relative to Cast-iron Beams.

I have probably bestowed more attention to this part of the subject than may at first sight appear necessary. It must, however, be borne in mind, that many serious accidents have occurred from an ignorance of the principles on which cast-iron beams and girders are constructed ; and I may perhaps be permitted to hope that the present investigation will not be without its use in those constructions, in ensuring economy of material and greater security in the erection of buildings, demanding the most careful consideration on the part of the architect and the engineer.

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* For an account of the dangerous nature of a beam trussed in this manner, see Appendix No. III.

Should these objects be accomplished, and more correct views of the principles of construction be established, I shall consider myself amply recompensed for the time and trouble which I have expended upon the investigation of the subject.

Before entering upon another important division of the inquiry, that of the strength of wrought-iron tubular girders, as applied to bridges, I shall shortly advert to the application of *wrought-iron beams* for the support of the floors of buildings, and to other purposes requiring solidity and security from fire. To the investigation of this part of the subject we bring with us a large, amount of existing knowledge of form and construction, which for the last thirty years has guided our efforts in this department of practical science. To Mr. Hodgkinson's able and conclusive inquiry into the strength of cast-iron beams, we are indebted for many useful formulæ and other aids in the art of construction. Cast iron cannot, however, be depended upon, even in the best forms, for several reasons—viz. unequal contraction in the cooling of the metal, the brittle nature of the material, imperfections and flaws in the castings, and its liability to break without warning.

As regards the first point, we labour under great uncertainty in consequence of the 'shrinkage' or contraction of metals during the process of cooling. A casting, even when well proportioned, will suddenly 'snap' without any apparent cause; exposure to rain, or intense frost during the night, not unfrequently produces fracture; and on these occasions rupture takes place with a loud noise, like the report of a pistol. On minute examination the injury is at once seen to have arisen from the presence of an immense tensile strain in the immediate vicinity of the fracture, which is generally found to be greatly enlarged, and an enormous force is required to bring the parts again into contact. This unequal and dangerous force of tension, existing within the casting itself, appears to me to be produced by one of two causes,—either from unequal rates in the time of cooling, whereby the crystalline process is seriously deranged, or from imperfect mixture of the metals, whence the 'shrinkage' is greater in one part than in another, and from which would follow unequal degrees of tension of the parts. Great care should therefore be observed in castings; it should be seen that the metals are well mixed, and that the moulds and patterns are so proportioned as to insure uniformity in the rate of cooling. These are practical operations of some importance, and the moulds, after running, should be covered closely up, and as much time as possible given for attaining, by a slow rate of cooling, a greater degree of perfection in crystalline structure.*

The second cause of danger is, that all crystalline bodies are of a more brittle and uncertain character than those which are of a fibrous structure; and as wrought iron possesses more ductility, and partakes in a greater degree of the latter quality, it is better qualified to sustain heavy weights and shocks than cast-iron; and its high powers of resistance to a tensile strain renders its application in the constructive arts an object of primary importance to all those connected professionally or otherwise with the erection of buildings. The superiority of its resistance to tension is not however its only recommendation, as the new forms and conditions under which it can be manufactured and applied, in position and distribution, to resist compression, is another powerful recommendation of it as a safer and lighter substitute for cast iron. Another defect of cast iron is the impossibility of discovering imperfections which may lie concealed under the surface of a casting, and which frequently baffle the scrutiny of the keenest observer. These defects are by no means uncommon;

---

* Tredgold appears to have been perfectly aware of the danger resulting from unequal cooling. In his remarks on the quality and appearance of the metals, in his treatise on the strength of cast iron, he states, p. 8: 'That the utmost care should be employed to render the iron in each casting of a uniform quality, because in iron of different qualities the "shrinkage" is different, which causes an unequal tension among the parts of the metal, impairs its strength, and renders it liable to sudden and unexpected failure.' At another part he observes: 'I must not omit to remark, that cast iron, when it fails, gives no warning of its approaching fracture, which is its chief defect when employed to sustain weights or moving forces.'

and repeated instances have occurred wherein castings, presenting every appearance of perfection, have been found to contain the elements of destruction, either in concealed air-bubbles, or in the infusion of scoriæ, which had been run into the moulds, and skimmed over by a smooth covering of apparently sound iron. Now, this can never occur in the wrought-iron beam, as the different processes of manufacture, such as puddling, forging, piling, and rolling, are sufficient to cause any imperfection, calculated to endanger the soundness of the plates, to be detected. It will, however, sometimes occur, that minute particles of scoriæ will be inserted between the laminæ or bars from which the plates are rolled; but this does does not materially affect the strength, excepting only in the case of boilers, where they form blisters when exposed to intense heat. In the formation of beams these defects are of less consequence, as they do not seriously impair their strength.

On the influence of Time and Temperature on the Strength of Cast-iron Bars.

Before closing our remarks on cast-iron beams, it may be advisable to introduce a few experimental facts in connection with two very interesting questions on the strength of materials, involving considerations of great importance, viz. the influence of time and of temperature on cast iron, or the extent to which these agencies respectively affect its power of resistance to a force tending to sever or rupture its parts. These questions occupied my attention some years ago, and considering that they bear directly upon the question now before us, it may be instructive if we give a few extracts from the experiments,* which were then considered of great value, and which led to several important results.

It has always been a question of doubt how far a body, say cast iron, can be loaded, without impairing its powers of resistance. This question is still imperfectly understood, as all bodies are surrounded with many causes of deterioration and disturbance, which affect their permanent condition, and which lead by slow but certain degrees to their ultimate destruction. Meteorological influence, temperature, and time, are all elements apparently at work in that direction; and it is curious to ascertain which of them has the greatest effect upon the stability of a material so extensively applied to the arts of construction as cast iron. In order to solve this problem, I entered in the year 1837 upon a series of experiments, which extended through a course of eight years; and as these experiments affect formerly acknowledged theories on the strength of materials, it may be essential not only to state the results, but to apply such deductions as were derived from the experiments as they were recorded from time to time.

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* For a more detailed description of the effects of time and temperature upon cast iron, the reader is referred to my report in the sixth volume of the 'Transactions of the British Association for the Advancement of Science.'

Effects of Time.

In the report read before the British Association for the Advancement of Science, the following observations on the effects of time on loaded bars occur.

In former experiments on the transverse strength of cast iron, it has been assumed that the elasticity remained perfect to the extent of at least one-third of the breaking weight, and that it should never be loaded with more than that weight. This assumption, which has been attempted to be proved by Tredgold, has gained considerable credence; but, so far as I can perceive, there appears to be no ground for such an opinion. In some early experiments on the comparative value of hot and cold blast iron, it was observed that in most cases the elasticity was considerably injured with one-fifth or one-sixth of the breaking weight. This fact was of such importance as to induce me to pay particular attention to the set, as indicated by the preceding experiments, and also to note the defects of elasticity in those that follow, up to the time the weights became permanent upon the bars. From the method thus pursued, it will be seen that the value of the set has been given with the deflections, at regular intervals, from the commencement of the experiments to the time of fracture, and the connections between the weights, deflections, and sets, will therefore in all probability be better observed.

We have already observed that Tredgold and others who have written on this subject have stated, that what was called the limit of elasticity attained by a transverse strain was one-third the breaking weight, or, in other words, it did not take a permanent set with less than that weight. Experiments by both Mr. Hodgkinson and myself have shown this to be an erroneous conclusion, as we found the elasticity injured with one-tenth, and as low as one-twelfth, the breaking weight. With these comparatively slight loads a permanent set was observed, which was never again restored. These defects, however, did not injure the bearing powers of the beams; on the contrary, it had the effect of increasing their powers of resistance by a tendency to equalise the strain over the whole of the molecular construction.

The early period at which the elasticity became injured caused an additional series of experiments to be instituted, for the purpose of ascertaining whether or not such injury to the elasticity would, with the load continued, ultimately break the bar. This important question could only be determined by experiment.

The inquiry, therefore, resolved itself into the question — to what extent cast iron could be loaded without endangering its security? This was a question of great importance; one which involved considerations of much interest, such as the stability of bridges, warehouses, factories, and other constructions in which cast iron is employed.

In computing the bearing powers of cast iron, it has always been considered unsafe to suppose it loaded beyond one-third of its breaking-weight, and, in order to be on the safe side, this is not an unwise precaution; but in some cases, such as bridges, beams for warehouses, &c., it is desirable to calculate only one-fifth or one-sixth of the breaking-weight, as the material may be subjected to accidental strain, arising from the force of impact, or from other forces acting unfavourably upon it. Taking the experiments, however, as a criterion, it will be found that out of ten rectangular bars, each 1 inch square, of cold and hot-blast Coed-Talon iron, certain results were obtained after the bars had been subjected to the following loads, as permanent weights suspended from the middle of each bar.

The ten bars were loaded with weights varying from about one-half to nine-tenths of their breaking weights, as given in the following table:—

Table I.—Table of Coed-Talon Rectangular Bars (4 feet 6 inches between the Supports) loaded with different Weights for determining the Changes (if any) which take place during Indefinite Periods of Time; the Mean of the Breaking Weights of each Description of Iron having been previously ascertained to be, for the Cold Blast 508 lbs., and for the Hot Blast 484 lbs.

No. of bars.	Permanent load in lbs.	Mean breaking weight in lbs.	Ratio of breaking weight to load.	Remarks.
1	280	508	1 : ·551	Cold-blast iron
2	336	508	1 : ·661	
3	392	508	1 : ·771	
4	448	508	1 : ·881	
5	448	508	1 : ·881	
6	280	484	1 : ·578	Hot-blast iron
7	336	484	1 : ·694	
8	392	484	1 : ·805	
9	448	484	1 : ·925	
10	448	484	1 : ·925	

From the above will be seen the nature of the experiments which were instituted for the purpose of ascertaining, by an exceedingly minute scale, the increase of deflection which from time to time took place in the bars. If that increase was progressive, it was then inferred that rupture must at some time or other (however remote) take place; if otherwise, that a new arrangement of the parts under strain had been effected, and that they had thus become fixed at a power of resistance equivalent to the load.

The results from March 1837 to April 1842 were as follow:—

Table II.—Results on Bars No. 2 and No. 7, loaded with 336 lbs.

Temperature.	Date of observation.	Cold blast, deflection in inches.	Hot blast, deflection in inches.	Rates of increase.
.	March 11, 1837 .	1·270	1·461	Previous to the time of taking the deflection in November and April the hot-blast bar had been disturbed.
78°	June 3, 1838 . .	1·316	1·538	
72°	July 5, 1839 . .	1·305	1·533	
61°	June 6, 1840 . .	1·303	1·520	
50°	Nov. 22, 1841 . .	1·306	1·620	
58°	April 19, 1842 . .	1·308	1·620	
	Mean . . .	1·301	1·548	

The above experiments show a mean increase in the deflections of the cold-blast bar during a period of five years of ·031, and of ·087 on the hot-blast bars.

TABLE III.—RESULTS ON BARS NO. 3 AND NO. 8, LOADED WITH 392 LBS.

Temperature.	Date of observation.	Cold blast, deflection in inches.	Hot blast, deflection in inches.	Remarks.
. .	March 6, 1837 . .	1·684	1·715	
78°	June 23, 1838 . .	1·824	1·803	
72°	July 5, 1839 . .	1·824	1·798	
61°	June 6, 1840 . .	1·825	1·798	
50°	Nov. 22, 1841 . .	1·829	1·804	
58°	April 19, 1842 . .	1·828	1·812	
53°	Mean . . .	1·802	1·788	

During five years the deflection in the cold-blast bars has been rather more than in the hot-blast, the increase being as 1·802 to 1·788; whereas in the former table, with lighter weights, the increase was on the other side, or as 1·301 in the cold-blast to 1·548 in the hot-blast bars.

Nevertheless, the deflections in this case indicate, as before, a steady increase of ·118 for the cold blast, and ·073 for the hot blast.

TABLE IV.—RESULTS ON BARS NO. 4, 5, 9, AND 10 LOADED RESPECTIVELY WITH 448 LBS.

Temperature.	Date of observation.	Cold blast, deflection in inches.	Hot blast, deflection in inches.	Remarks.
. .	March 6, 1837 . .	1·410	. .	Both the hot-blast bars broke at once with 448 lbs., and one of the cold-blast bars broke after sustaining the weight thirty-seven days.
78°	June 23, 1838 . .	1·457	. .	
72°	July 5, 1839 . .	1·446	. .	
61°	June 6, 1840 . .	1·445	. .	
50°	Nov. 22, 1841 . .	1·449	. .	
58°	April 19, 1842 . .	1·449	. .	
53°	Mean . . .	1·442	. .	

The mean increase of the deflections in this case is ·032; which, it will be observed, is much less than that exhibited in the former table with weights of 392 lbs., and less than the deflections on the hot blast, which were ·073 with the same weight continued as a permanent load.

Viewing the whole of these experiments in relation to the solution of a problem affecting the laws which regulate the resistance of bodies to continuous strain, it is important to observe how admirably the cohesive powers of matter adapt themselves to circumstances, and with what tenacity they resist forces tending to dissever and rupture their parts.

It is still a question for consideration how far this power extends, and whether or not bodies, when loaded within even $\frac{1}{1000}$ part of their breaking-weight, would sustain the load for ever, provided that no disturbing cause were present to produce fracture.

I am strongly inclined to think that such would be the case, notwithstanding the fact that the whole of the loaded bars exhibit a progressive increase of deflection, which fact I am disposed to attribute to the vibrations continually going on in the building where the bars were fixed, and to those atmospheric changes, such as temperature, oxidation, &c., to which every description of material is subject.

In the experiments just enumerated one important fact has been fully established, namely, that a continued and strictly permanent pressure, even when about to produce fracture, obeys a very different law to that which obtains with a continual increase and diminution of pressure, producing a disturbing force on all the parts under strain, and thus, by a continued series of alternations, eventually destroying the powers of resistance.

In the former case the load, however near it approximates to that which is necessary to fracture, remains permanently fixed; whereas in the latter, the changes, however minute, will, if continued long enough, ultimately lead to destruction. Mr. Hodgkinson's experiments, as well as my own, lead to this conclusion; and I have no doubt that any load (however small) producing a permanent set upon a bar will, if taken off and replaced a sufficient number of times, eventually break it.

For example, let us take the bars supporting the lightest weight, 280 lbs., and suppose that the load to the extent of 200 lbs. is removed, and then replaced at intervals of thirty seconds, it is evident that this change, often repeated, will in the end destroy the cohesive powers of the bar, either in the lower part of its crystalline extended section, or in that of the upper, where the crystals are compressed; or, what is more probable, the destructive process would be progressive in a given ratio in the upper and lower sections of the respective means of compression and extension. This constant movement or sliding of the atoms or parts of crystalline, as well as fibrous bodies, is therefore the cause of breakage; and—assuming a change to take place in the molecules of the body—however slight the strain may be when first applied in one direction, and then changed to another direction, it only becomes a question of time how long the body will bear these continued repetitions before rupture takes place: sooner or later fracture must occur.

In illustration of the above we may refer the reader to the Commissioner's Report on the Application of Iron to Railway Structures, in which he will find the results of a long series of experiments bearing upon this subject. It will be sufficient here to say that bars of cast iron 3 inches square, or less, were placed upon supports varying from 4½ feet to 14 feet asunder. These were struck upon their side by a heavy ball suspended like a pendulum from the roof, by a wire 18 feet long, the magnitude of the blow being varied as occasion required. The general result obtained was that, when the blow was powerful enough to bend the bars through one-half of their ultimate deflection (that is to say, the deflection which corresponds to their fracture by dead pressure), one bar only was able to stand 4,000 of such blows in succession, some giving way with less than 130 blows. But all bars when sound resisted the effects of 4,000 blows, each bending them through one-third of their ultimate deflection. The following table gives a summary of the mean results:—

No. of Experiments.	Assigned deflection in terms of ultimate deflection.	Mean No. of blows struck.	Remarks.
7	$\frac{1}{3}$	4,000	Not broken.
1	$\frac{5}{12}$	1,350	Broken.
7	$\frac{1}{2}$	1,838	All broken but one. Five slightly defective.
1	$\frac{7}{12}$	3,700	Broken. Slightly defective.
5	$\frac{2}{3}$	178	All broken. One slightly defective.

In the above we have excluded those cases in which the bars were so defective as manifestly to influence the result. In regard to the bar which stood 4,000 blows bending it through $\frac{1}{2}$ its deflection, we may observe that it was a small bar only 1 inch square and 4 feet 6 inches between supports, and hence may possibly have been somewhat tougher than the larger bars.

I am further confirmed in the truth of these observations by some experiments now in progress, which bear so forcibly upon this peculiar aspect of the strength of materials, that I am induced to notice them in this place. I do so in order that we may become fully aware of the principles on which the security as well as the durability of the material depends. It is important that all these facts should be generally known, as there is a wide difference between the bearing powers of a beam exposed to changes of pressure, and one which has to sustain a perfectly quiescent and permanent load.

On the Effects of Temperature.

The principles of the effects of temperature upon cast-iron are pretty fully developed in my Report, published in the sixth volume of the Transactions of the British Association for the Advancement of Science. The experiments therein contained are very conclusive; and those on time would have been less satisfactory if those on temperature had been omitted.

In that communication it is stated that Rondelet, in his *Traité de Bâtir*, had given and collected results from experiments made by himself and others on the expansion of bodies from the effects of heat; but I believe I was the first to ascertain the strength of metallic substances at different temperatures; and although the effects of heat upon metals had not escaped the attention of philosophers, yet I am not aware that any writers on this subject have conducted their experiments in a way at all analogous to that under consideration.

Had time permitted, it was my intention to have pursued the experiments on temperature under a much greater variety of form and change. For example, it might have been desirable not only to load the bars until they were broken, but also to charge them with different weights, and by alternate heating and cooling to ascertain how the bars so charged were affected by the change. Such an extension of the experiments might have led to the development of some new law, and that more particularly as regards the changes produced by the alternate increase and decrease of temperature. On some future occasion I may perhaps have an opportunity of returning to these interesting inquiries; they involve considerations of great importance, as well in theoretical as in practical science, and I have no doubt that they would be found of great value in every case where the material is exposed to frequent changes of temperature. For the present, however, it will be sufficient to give the comparative results which were obtained from the experiments.

TABLE V.—COMPARATIVE STRENGTH AND POWER TO RESIST IMPACT OF THE COED-TALON HOT AND COLD-BLAST IRONS AT VARIOUS TEMPERATURES.

TRANSVERSE STRENGTHS.

Temperature.	Coed-Talon cold blast.	Coed-Talon hot blast.	Ratio.
	No. 2 Iron.	No. 2 Iron.	
Fahr.	lbs.	lbs.	
26°	851·0	823·1	1000 : 967·2
32°	940·7 958·5 Mean 949·6	933·4 906·0 Mean 919·7	1000 : 977·6
190°	743·1	823·6	1000 : 1108·
Red in dark .	723·1	829·7	
	No. 3 Iron.	No. 3 Iron.	
	lbs.	lbs.	
212°	905·0 943·6 Mean 924·3	818·4	1000 : 885·4
600°	909·3 1157·0 „ 1033·1	834·1 917·5 Mean 875·8	1000 : 847·7

POWER TO RESIST IMPACT.

Temperature.	Coed-Talon cold blast.	Coed-Talon hot blast.	Ratio.
	No. 2 Iron.	No. 2 Iron.	
Fahr.	lbs.	lbs.	
26°	349·8	340·8	1000 : 974
32°	360·3 401·5 Mean 382·4	436·9 383·2 Mean 395·0	1000 : 1032·9
190°	223·7	298·9	1000 : 1336

Modulus of Elasticity in Pounds for a Base of 1 inch square.

Temperature.	Coed-Talon cold blast.		Coed-Talon hot blast.	
	No. 2 Iron.		No. 2 Iron.	
Fahr.	lbs.		lbs.	
26°	12,994,400		14,267,500	
32°	13,506,700 15,148,200	Mean 14,327,450	13,723,500 14,283,200	Mean 14,003,350
190°	14,398,600		13,869,500	

In pursuing the investigations, it unfortunately happened that the stock of No. 2 iron became exhausted, a circumstance which intercepts the comparison from 6 degrees below the freezing-point to that of melted lead. The No. 3 should have been broken at all degrees of temperature, in order to ascertain the loss of strength as the heat was increased. It was not, however, accomplished; and from this circumstance the comparison only holds good between the two qualities No. 2 and No. 3 from the boiling-point of water, or 212°, up to 600° of Fahrenheit. In the No. 2 iron it will be observed that the strength continued to diminish as the temperature increased; whereas in No. 3 it increased, as shown in the Table, from 924·3 to 1033·1, which can only be accounted for from the irregularity and greater rigidity of that description of iron. On the whole, we may infer that cast iron of average quality loses strength when heated beyond a mean temperature of 120°, and that it becomes insecure at the freezing-point, or under 32° Fahrenheit.

On the Comparative Strength of Iron after Successive Re-meltings.

Some curious and exceedingly interesting facts were arrived at in a series of experiments to determine the maximum from which iron begins to descend in the scale of strengths. This inquiry was undertaken at the request of the British Association for the Advancement of Science, and was intended to determine the variations in the strength of iron when many times re-melted.

In preparing the iron, coke, and flux requisite for ensuring sound castings, it was found necessary, for the sake of comparison, to have them cast under circumstances precisely the same, and in order to ensure as nearly as possible perfect uniformity in the castings, a furnace was prepared for the express purpose, and from 18 cwt. to a ton of No. 3 Eglinton hot-blast iron was melted, and run into bars and pigs, with 588 lbs. of coke and 224 lbs. of lime as a flux. The proportions of coke and flux were carefully observed throughout all the subsequent meltings. The materials were accurately weighed every time the furnace was charged, and each charge was melted under the same circumstances, and as nearly as possible with the same quantity of blast. In the first melting, three or four bars, each 5 feet long and 1 inch square, were cast, and the remainder of the iron run into pigs and preserved for re-melting, along with the fractured bars used in the first experiment. In the succeeding experiments, the bars and pigs were prepared and re-melted in the same way; and thus, by a continual succession of meltings, the constant reproduction of the same metal was carefully preserved, and that under the same circumstances of fusion, as respects coke and flux, as those previously melted, until the whole of the metal was exhausted.

The pig iron used in these experiments was No. 3 Eglinton hot blast. From its blue tinge and large crystalline structure, it had the characteristics of No. 1 more than No. 3; and judging from its appearance, it indicated a ductile and superior quality of iron; probably of more value for its working properties than its powers of resistance to strain. This property in the metal was not, however, objectionable, as it enabled us to continue the experiments through a much larger series of meltings, and was probably as good a selection as could have been made for such an inquiry.

Results of Experiments on the Transverse and Crushing Strength of re-melted Iron.

Bars cast to be 5 feet long and 1 inch square. Distance between supports 4 feet 6 inches.

No. of meltings.	Specific gravity.	Mean breaking weight of bars exactly 1 in. square in lbs.	Mean ultimate deflection in inches.	Power to resist impact in lbs.	Resistance to compression in tons per square inch.
1	6·969	490·0	1·440	705·6	44·0
2	6·970	441·9	1·446	630·9	43·6
3	6·886	401·6	1·486	596·7	41·1
4	6·938	413·4	1·260	520·8	40·7
5	6·842	431·6	1·503	648·6	41·1
6	6·771	438·7	1·320	579·0	41·1
7	6·879	449·1	1·440	646·7	40·9
8	7·025	491·3	1·753	861·2	41·1
9	7·102	546·5	1·620	885·3	55·1
10	7·108	566·9	1·626	921·7	57·7
11	7·113	651·9	1·636	1066·5	69·8
12	7·160	692·1	1·666	1153·0	73·1
13	7·134	634·8	1·646	1044·9	66·0*
14	7·530	603·4	1·513	912·9	95·9
15	7·248	371·1	0·643	238·6	76·7
16	7·330	351·3	0·566	198·5	70·5
17	Lost.	—	—	—	—
18	7·385	312·7	0·476	148·8	88·0

The above constitutes the results as obtained from the whole series of meltings, and it will be observed that the maximum of transverse strength, elasticity, &c., is only arrived at after the metal has undergone twelve successive meltings. It is probable that other metals and their alloys may follow the same law, but that is a question which has yet to be solved, and that probably by a series of experiments which would require a considerable amount of time and labour to accomplish.

In further proof of the value of re-melting iron when a rigid strong metal is required, we may annex the following summary of some experiments recently conducted by Major Wade, of the Ordnance Department of the United States Army. We may observe that in these experiments a very large quantity of iron was melted at a time, and was kept in fusion for periods varying from two hours to two hours and a half, and hence the metal approached its maximum strength after much fewer re-meltings than in the last experiments. At the same time it would have been very interesting had the fusions been repeated a fifth, sixth, or seventh time, in order to have determined more accurately the precise point at which the iron began to deteriorate in strength.

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* The cube did not bed properly upon the steel plates, otherwise it would have resisted a much greater force—probably 80 or 85 tons per square inch.

Grades of Greenwood iron tested.	No. of fusion.	Test of Samples.			
		Proof bars.		Gun-heads.	
		Density.	Tenacity in lbs. per sq. in.	Density.	Tenacity in lbs. per sq. in.
No. 1 . .	1st	7·032	15,129	—	—
No. 2 . .	1st	7·153	27,153	—	—
No. 3 . .	1st	7·230	34,923	—	—
No. 1 . .	1st	7·032	15,129	—	—
	2nd	7·086	21,344	—	—
	3rd	7·198	30,107	7·090	25,716
	4th	7·301	35,786	7·257	33,815*
No. 1 of 3rd fusion, and No. 3 of 2nd	2nd &	7·259	36,916	7·169	28,910
	3rd	7·270	39,373	7·221	29,751
Nos. 1 and 2, mixed . .	2nd &	7·170	27,588	—	—
	3rd	7·272	40,897	7·228	32,421
Nos. 1, 2, and 3, mixed . .	2nd &	7·251	37,789	—	—
	3rd	7·340	32,485	7·260	38,093

'The foregoing statements exhibit the qualities of the several grades of the Numbers 1, 2, and 3 iron, made at the same furnace, from the same ores, and they show how each are modified by different fusions at the foundries.

'In the No. 1, which is the lowest grade of soft gray iron, the mean of numerous trials is, in the proof bar samples of first fusion, a density of 7·032 and a tenacity of 15,129 lbs. The same iron, identically, after being three times re-melted, gives in the same sort of proof bar samples of fourth fusion a density of

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* Speaking of some of the specimens of No. 1 iron, Major Wade states, that 'the iron, although of the 4th fusion, was still of a medium soft gray, and would no doubt have been further improved by another melting, which would have made its fifth fusion.'

7·301 and a tenacity of 35,786 lbs.; being an increase of density equal to about 17 lbs. in the cubic foot, and an increase of tenacity in the ratio of 100 : 236. This change in these qualities of iron is effected solely by re-melting, and exposure to heat while in fusion. A similar change appears in the gun-head samples, made from the same iron, at the third and fourth fusions.

'The above refers to the No. 1 grade of iron exclusively. And it will be seen by the foregoing table, that the different grades 1, 2, and 3, vary very materially in quality, in the original pigs of first fusion. By adding to the No. 1 of third fusion a portion of No. 3 of second fusion, a considerable improvement is made; and by uniting Nos. 1 and 2, in equal parts, both of third fusion, still better results are obtained in both proof bars and gun-heads. But when Nos. 1, 2, and 3, are united in third fusion, the highest point of strength in gun-heads is attained, viz. 38,000 lbs., which is the mean of the last four guns cast, all of which are of this composition.

'The softest kinds of iron will endure a greater number of meltings with advantage than the higher grades. The maximum tenacity of proof bars is attained in the No. 1 iron, at the fourth fusion; in the Nos. 1 and 2, mixed, at the third fusion; and in Nos. 1, 2, and 3, mixed, at the second fusion. At the third fusion of the latter composition the tenacity of the proof bars diminished, and at the same time it increased in the gun-heads.

'It appears that when iron is in its best condition for casting into proof bars of small bulk, it is then in a state which requires an additional fusion, to bring it up to its best condition for casting into the massive bulk of cannon. This is distinctly shown in the castings composed of the Nos. 1, 2, and 3 iron mixed. In the other compositions of lower grades the guns were cast at the same meltings, which gave the highest tenacity in the proof bars. And in these cases the guns were of comparatively low tenacity.

'The mixtures of Nos. 1, 2, and 3, in second and third fusion, and of 1 and 2 in third fusion, are examples of this. And from the low densities of the gun-heads in these cases, there is good reason to suppose that an additional fusion of this iron would have augmented the tenacity of the gun-heads, while it would have diminished that of the proof bars. It may therefore be assumed as a rule not liable to material error, with this Greenwood iron at least, that the strength of proof bars at any fusion may be taken as an approximate measure of the strength of gun-heads, made of the same iron at the next fusion. In selecting and preparing iron for cannon, we may therefore proceed by repeated fusions, or by varying the proportions of the different grades, until the maximum tenacity in proof bars is attained; the iron will then be in its best condition for being again melted and cast into cannon.

'It will be seen, that the density of the several grades of iron, Nos. 1, 2, and 3, is in the order of their respective numbers, and that all kinds increase at each additional fusion; and, also, that the density is greater when cast into small proof bars, which cool quickly, than when cast into large masses which cool slowly. It appears also that the tenacity increases quite uniformly with the density, until the latter ascends to some given point; after which an increased density is accompanied by a diminished tenacity. The turning point of density at which this Greenwood iron attains its greatest tenacity appears to be about 7·27. At this point of density or near it, whether in proof bars or gun-heads, the tenacity is greatest. No example of density between 7·272 and 7·301 occurs in the foregoing table, but in every instance where the latter or any higher density is exhibited, it is accompanied by a diminished tenacity.

'As the density of the iron is increased, its liquidity when melted is diminished. This causes it to congeal quickly, and to form cavities in the interior of the casting. In the three guns last cast, Nos. 420, 427, and 428 [Iron Nos. 1, 2, and 3, mixed, third fusion], the iron was carried up to its highest limit in this respect. Cavities appeared in the gun-heads, and in such parts of the muzzle and bore as nearly proved fatal to the castings.'*

The results noted by Major Wade exhibit greater increase of strength than in the previous experiments on the Eglinton iron, where we had an increase in the transverse breaking weight from 401 lbs. at the first, to a maximum of 692 lbs. at the twelfth melting, or in the ratio of 1 : 1·72. Reasoning from these facts, there cannot exist a doubt as to the advantages to be derived from the re-melting of iron, whenever castings of great resisting powers and tenacity are required.

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* Reports of Experiments on Metals for Cannon. Philadelphia, 1856.

Mixtures of Iron.

On the subject of the mixtures of the different kinds of British iron, for the purpose of producing castings suitable to the purposes for which they are intended, we have no fixed or determined rule from which we can obtain anything like correct results. Every iron-founder appears to exercise his own judgment in those matters; and it is very difficult to obtain castings under conditions where the proportions are specified, unless prepared under a strict and rigid surveillance. Iron-founders, managers, and furnace-men appear to work under the impression of the non-importance of attention to the mixture of metals; and hence arise all the anomalous conditions of soft and hard, strong and weak irons, and many other disqualifications which might easily be avoided by closer attention to the quality and due proportion of each particular iron, and the quantity of carbon and flux used in its liquefaction. All these are considerations of great importance in the art of founding, and we have yet much to learn in the preparation of metals, as well as in the manipulative art of moulding, and the necessary process of ventilation—a process which requires no inconsiderable degree of thought and skill.

It has always been an opinion that cast iron is improved by mixing; and doubtless this is the case, as we have almost every variety in the pig—soft, hard, ductile, rich, and poor, as well as the white, blue, grey, &c., irons, all of which are adapted in combination to form almost any description of metal required in the useful arts. They are, moreover, calculated to effect great improvement in the quality of the castings, and form compounds which, with proper care, may be varied according to the uses for which the castings are intended. In the case of beams and bridges, a mixture of two-thirds strong Welsh No. 3, a portion of Scotch or Staffordshire No. 2, and a proportion of old iron, will form a good mixture. Other strong irons may be used, such as the finely granulated Scotch or Staffordshire No. 3; a small portion of No. 4, and about one-fourth or one-fifth of Hæmatite, answers the same purpose. These compounds are of great value when it is essential to have strong metal; but in ordinary cases, the mixture may be left to the discretion of those accustomed to the management of the cupola and the furnace.

As an example of the method of mixing iron from different furnaces, and of the results of different modes of treatment, the following table will be of interest. It contains the specific gravity and tensile strength of specimens of iron, cut out from five experimental 24-pounder guns, prepared by the Author, and tested under the direction of Col. Wilmot of the Royal Arsenal, Woolwich.

Mark on gun.	No. of specimen.	Position in the gun from which the specimens were cut.	Specific gravity.	Tensile Breaking weight per sq. inch in lbs.	Mean.		No. of rounds fired before bursting.
					Specific gravity.	Breaking weight per square inch in lbs.	
A	1	Breech	7·2383	30,060	7·2105	28,516	33
	2	Muzzle	7·1827	26,971			
B	3	Breech	7·2290	29,426	7·2325	27,219	32
	4	Muzzle	7·2360	25,013			
C	5	Breech	7·0976	18,657	7·0863	18,101	17
	6	Muzzle	7·0751	17,544			
D	7	Breech	7·2191	26,218	7·2032	25,954	31
	8	Muzzle	7·1874	25,690			
E	9	Breech	7·2494	27,649	7·2441	28,516	33
	10	Muzzle	7·2388	29,382			

The mixtures which produced these exceedingly strong specimens will be seen from the following table:—

	Gun A.	Gun B.	Gun C.	Gun D.	Gun E.
Name and quality of Iron.	Cast in usual way, 3 ft. 6 in. head.	Remelted once, and then cast as Gun A.	Run from cupola and melted by desulphurised coke, 3 ft. 6 in. head.	Cast under dead-head pressure of 17 ft. 3 in.	Remelted, and then cast under dead-head pressure of 17 ft. 3 in.
	cwt. qrs.	cwt. qrs.	cwt. qrs.	cwt. qrs.	cwt. qrs.
Blaenavon, No. 1	1 1	2 0	1 1	1 1	2 0
Blaenavon, No. 2	11 2½	15 0	11 2½	11 2½	15 0
Lilleshall, No. 2	24 0½	32 0	24 0½	24 0½	32 0
Pontypool, No. 3	11 3	15 0	11 3	11 3	15 0
Blaenavon, No. 3	19 3	26 0	19 3	19 3	26 0
Lilleshall, No. 3	—	—	6 2	—	—
Scrap from gun .	5 0	—	10 0	5 0	—
Gun head . . .	11 2	—	—	11 2	—
	85 0	90 0	85 0	85 0	90 0

From the above table it will be seen that guns A and E stood the same number of rounds, but from the condition of the guns after bursting, from the microscopic appearance of the fracture, and from other circumstances, it was evident that E was of decidedly the best quality of iron. It will be observed that this gun was of maximum density, and was cast under a pressure of 53 lbs. to the square inch. It was of the same tensile strength as gun A, namely 28,516 lbs. per square inch, but of a higher specific gravity, due probably to its having been cast under a head of 17 feet 3 inches, instead of 3 feet 6 inches. It exhibited a closer and more finely granulated structure than any of the others, and doubtless in case of repetition would have indicated results as favourable in its powers of resistance to strain as in its density and closeness of texture.

It will be necessary to observe that the weight of shot and charge of powder was increased every five rounds, beginning at 8 lbs. of powder and two 24-pounder shot, and increasing to 14 lbs. of powder and seven shot, = 168 lbs., at the thirty-third or last round. The guns A and E supported a total of 364 lbs. of powder, 3,120 lbs. of shot, and 91 wads, before bursting.

In selecting metal for casting ordnance it is necessary that it should be of the greatest purity, and as free as possible from phosphorus and sulphur; its specific gravity should not be less than 7·0 or 7·2; but density is not always an indication of strength, as we find in many cases great tenacity accompanied with a comparatively low density, and *vice versâ*; but generally, when the metal is free from impurities, the strengths are in the ratio of the specific gravities. The same remarks apply to all other descriptions of cast iron, in their application to purposes where great strength is required.

In this stage of the inquiry it will be proper to notice a mixture which, above all others, gives indications of superior strength. In the summer of 1847, Mr. Owen, the Supervisor of Metal to the Admiralty, was kind enough to forward to me a copy of his experiments upon Mr. Morries Stirling's toughened iron, whereby the tensile strength of cast iron (according to Mr. Morries Stirling's statement) is nearly doubled.

This process of toughening cast iron is by an admixture of wrought iron, fused simultaneously along with pigs in the cupola or air furnace. Mr. Owen gives the results from both processes, and the experiments being somewhat analogous, I have taken the liberty of referring to those which bear more immediately upon the subject of the present inquiry. Mr. Owen's experiments were made on a large scale. The girders were 17 feet long, and 16 feet between the supports, as in the annexed section (taken through the middle, where the weight was laid on).

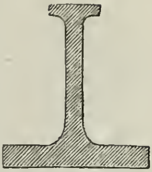

Fig. 31.

The girders were constructed upon Mr. Hodgkinson's principle, and weighed about 15 cwt. each, and their computed breaking weight was 39½ tons.

The girders were made of cast iron, prepared according to Mr. Morries Stirling's mixture, as follows:—

Cupola.

	cwt.	Breaking weight.
Russell's Hall hot-blast No. 2 Staffordshire .	15	50½ tons.
Prior Field No. 2 Staffordshire	20	
Wrought-iron scrap	6	

Air Furnace.

The same mixture as above sustained 51½ tons, giving an increase of strength of 2 per cent. in favour of the air furnace.

The experiments with seven different castings made entirely from the Russell's Hall No. 2, Madeley Wood No. 3, Colebrook Vale No. 3 (Welsh), and Calder No. 1 (Scotch), give an average of 33¼ tons as the breaking weight. Comparing the results of these experiments, we find the value, as regards strength, to be as 33·25 to 51·5, being the ratio of 1 : 1·55.

It will not be necessary to extend these examples beyond the following list of comparative results, as obtained from the series of experiments made upon each kind of iron:—

UNPREPARED CAST IRON.	Breaking weights.
1	30 tons.
2	35 ,,
3	33 ,,
4	34 ,,
5	33¼ ,,
6	34½ ,,
7	43¾ ,,
8	46½ ,,
9	47 ,,
10	47¼ ,,
11	38½ ,,
12	36½ ,,
13	38½ ,,
Mean tons	38·3

TOUGHENED CAST IRON.	Breaking weights.
1	52½ tons.
2	50½ ,,
3	48 ,,
4	52 ,,
5	52½ ,,
6	60½ ,,
7	52½ ,,
8	50½ ,,
9	56 ,,
10	48½ ,,
11	52 ,,
Mean tons	52·3

The ratio given by the above table is lower than that given by the first, being as 1 : 1·36; but it is nevertheless sufficient to show that the mixture decidedly improves the bearing powers of a casting when subjected to a transverse strain.

The following results were obtained by the Commissioners on Railway Structures, and exhibit as forcibly as the experiments of Mr. Owen the advantage of a mixture of cast and wrought iron.

MR. MORRIES STIRLING'S IRON.

Description.	Transverse breaking weight.	Ultimate deflection.
	lbs.	inches.
Bars 9 feet between supports, and 2 inches square	2,174 from 4 Experiments on 2nd Quality	2·652
Ditto ditto ditto	1,491 from 2 Experiments on 3rd Quality	2·179
Bars 4½ feet between supports, and 1 inch square	623 from 4 Experiments on 2nd Quality	1·482
Ditto ditto ditto	499 from 4 Experiments on 3rd Quality	1·302

The iron denominated 2nd Quality was composed of Calder iron No. 1 hot blast, and about 20 per cent. of malleable iron scrap; the 3rd Quality of Ley's No. 1 hot blast Staffordshire, and about 15 per cent. of common malleable iron scrap.

For comparison we annex some results obtained at the same time.

BLAENAVON IRON, No. 2.

Description.	Transverse breaking weight.	Ultimate deflection.
	lbs.	inches.
Bars 9 feet between supports, and 2 inches square . . .	1,338 from 6 Experiments .	3·0035
Bars 4½ feet between supports, and 1 inch square	440 from 3 Experiments .	1·779

That is, there is a mean increase of strength over the Blaenavon iron of 51 per cent. for the 2nd Quality, and of 13 per cent. for the 3rd Quality of Mr. Stirling's mixture. A similar large increase in the compressive and tensile strengths will be observed in the table, page 72.

Almost simultaneously with, or shortly after, Mr. Owen's experiments, I was called upon to witness some experiments on square bars, made by my friend Mr. Lillie, of Store Street, Manchester, which had an admixture of wrought iron, chiefly from turnings. They appeared to fuse and combine with cast iron in the cupola in various proportions; and having been present at some of the experiments, I can vouch for the great superiority of strength which Mr. Lillie's mixture seemed to indicate.

Mr. Lillie's experiments were made upon cast-iron bars 3 feet long, 1 inch square, and 2 feet 10 inches between the supports.

Number of Experiments.	Quality of Iron.	Breaking weights in lbs.	Deflection in inches.	Remarks.
1	Composed of Gartsherrie iron .	831	·625	
2	Composed of Mr. Lillie's Mixture .	1,346	·750	
3	Wrought-iron bar, same length, 1 inch square. .	1,008	·625	With an additional weight the bar was bent and destroyed, so as to render it incapable of bearing the load.

From the above table it appears that the mixed or toughened iron, as prepared by Mr. Lillie, is increased in its transverse strength more than one-third, as compared with cast iron, and nearly one-eighth as compared with wrought iron. It is, however, to be regretted that Mr. Lillie did not extend his experiments so as to give the comparative values of the different irons, by reducing the bars experimented upon to exactly one inch square. It would also be of great value to ascertain the relative tensile strain and crushing power of these mixtures, as there cannot exist a doubt as to the increased strength, either transverse or tensile, of cast iron, when fused with a due proportion of wrought-iron borings, turnings, or scrap iron.

Entertaining these views, it may be worthy of consideration how far it may be advisable to institute a series of experiments on the subject of these mixtures, and not only to determine accurately the comparative strengths of cast iron in combination with certain portions of wrought iron, but to extend our knowledge generally on a subject which is still exceedingly imperfect; in fact, such was our ignorance on this subject, that until Mr. Morries Stirling exhibited these properties of combination, it was supposed that wrought iron would not fuse and combine with cast iron.

Under circumstances where the toughened iron is not used, the following mixture will be found of great value in castings, such as girders for bridges, beams for buildings, &c., where rigidity and strength are required.

Low Moor Yorkshire, No. 3	30 per cent.
Blaina or Yorkshire, No. 2	25 ,,
Shropshire or Derbyshire, No. 3	25 ,,
And good old scrap	20 ,,
	100

The above mixture will make what may be considered castings of superior strength; but it seldom happens that this mixture can be obtained, on account of the high price of Low Moor iron; hence it is that we can rarely depend upon ironfounders for the introduction of the exact quantity necessary to produce the required results. These matters are generally left to subordinates, and either through ignorance, or to save themselves trouble, they almost invariably take what comes readiest to hand, and thus defeat, not only the calculation of the mathematician, but the hopes of those who confide in them.

There are other combinations or mixtures of iron which possess other properties besides those of strength, such as the Scotch and Staffordshire iron for light work and for castings for machinery; No. 1, or the richer description of iron, which is easily worked, and is more ductile than the strong Welsh irons.

In practice these mixtures require great consideration, as the success of the particular manufacture in some measure depends upon the castings produced. It would occupy too much time to enter upon all the questions connected with this subject. Suffice it to observe that, after having made the selection and determined upon the mixture, much even then depends upon the skill and care of the furnace-man, particularly in attending to the temperature of the furnace, and the degree of heat at which the metal is run into the mould.

All these are considerations which it would be superfluous at this time to discuss, but which enter largely into the formation of the crystalline structure, and which on no account should be neglected in the production of castings having for their object the union of the properties of fusibility and strength.

We may further observe that the anthracite is a strong description of iron, and will be found useful for mixing where rigidity and strength are regarded. Experiments on this iron will be found in the sixth volume of the second series of the Manchester Memoirs, and in the following tables.

The annexed table, pp. 73 and 74, the result of several years' labour, shows the transverse strengths, and many other properties, of nearly all the irons of the United Kingdom, and, with some other tables containing useful data, may appropriately close this part of the subject.

In the Commissioners' Report on Railway Structures we have a series of experiments on the transverse strength of cast iron, obtained from different iron-works; and as these experiments were undertaken, under the direction of Mr. Robert Stephenson, for the purpose of obtaining the best quality of iron for the castings of the High Level Bridge at Newcastle-on-Tyne, and since they disclose some new facts, we have less hesitation in giving them a place beside our own, and with others we have selected from different sources.

We have also selected a short abstract from a Report made to the United States Ordnance Department, on the 'causes which so materially affect the quality of gun-metal.' And as this report relates to the chemical constituents of different descriptions, it will supplement and complete our investigations on the qualities of cast iron.

EXPERIMENTS ON THE TRANSVERSE STRENGTH OF IRON, MADE UNDER THE DIRECTION OF ROBERT STEPHENSON, ESQ.

Bars 1 inch square, and 3 feet between Supports.

	Description of Iron.	No. of Experiments.	Mean breaking weight in lbs.	Mean ultimate deflection in inches.	Reduced breaking weight to bars 4ft. 6in. between supports.
Hot Blast.	Scotch	3	775	·63	516
	Coltness, No. 3	3	789	·73	526
	Langloan, No. 3	3	727	·66	484
	Omoa, No. 3	3	906	·82	604
	Omoa, No. 1	2	805	·75	536
	Redsdale, No. 3	3	1,014	·89	676
	Redsdale, No. 1	3	794	·745	529
	Redsdale, No. 1 (selected)	3	919	·94	612
	Tow-Law, No. 3	2	708	·73	472
Cold Blast.	Staffordshire, No. 3	3	873	·79	582
	Crawshay (Welsh), No. 1	3	873	·80	582
	Blaenavon, No. 1	3	754	·82	502
	Coalbrook Vale, No. 1	3	876	·71	584
	Coalbrook Vale, No. 3	3	897	·73	598
	Ystalyfera, No. 3, hot blast, anthracite iron	6	998	·80	665
Mixtures.	Ystalyfera, No. 3, hot blast Blaenavon, No. 1, cold blast	3	876	·82	584
	Garscube, No. 1, hot blast Redsdale, No. 3, hot blast	2	981	·83	654
	Garscube, No. 1, hot blast Redsdale, No. 3, hot blast	3	907	·80	604
	Dundyvan, No. 3, hot blast Coltness, No. 3, hot blast	3	824	·71	548
	Redsdale, No. 1, hot blast Clyde, No. 3, hot blast Coltness, No. 3, hot blast	3	859	·82	572
	Langloan, No. 3, hot blast Omoa, No. 1, hot blast Forth, No. 3, hot blast	3	829	·82	552
	Omoa, Blair, Clyde, Langloan, Forth, Coltness, all No. 3, hot blast	3	901	·75	600
	Scotch hot blast and scrap, ordinary foundry mixture for general purposes	3	879	·78	586

	Description of Iron.	No. of Experiments.	Mean breaking weight in lbs.	Mean ultimate deflection in inches.	Reduced breaking weight to bars 4ft. 6in. between supports.
Mixtures.	Carnbroe, No. 1, hot blast Redsdale, No. 3, hot blast	2	717	·59	478
	Carnbroe, No. 1, hot blast $\frac{2}{3}$ Redsdale, No. 3, hot blast $\frac{1}{3}$ Scrap (chiefly cold blast)	3	893	·67	595
	Crawshay (Welsh), No. 1, cold blast Coalbrookdale, No. 1, cold blast	4	855	·89	570
	Ystalyfera, No. 3, anthracite, 40 parts Redsdale, No. 3, hot blast, 40 parts Crawshay, No. 1, cold blast, 40 parts Blaenavon, No. 1, cold blast, 30 parts Coalbrookdale, No. 1, cold blast, 30 parts Scrap, selected clean, chiefly cold blast, 30 parts	3	1,058	·89	705*
	Ditto, second melting	3	524	·26	346†
	Ditto, cast from cupola	2	928	·80	618
	Ditto, cast from air-furnace	1	1,023	·94	682
	$\frac{1}{9}$ part Crawshay, No. 1, cold blast $\frac{1}{3}$ part Redsdale, No. 3, hot blast $\frac{5}{9}$ part Scotch, Nos. 1 and 3, hot blast	3	822	·87	548
	$\frac{1}{6}$ part Crawshay, No. 1, cold blast $\frac{1}{2}$ part Redsdale, No. 3, hot blast $\frac{1}{3}$ part Scotch, No. 1 and No. 3, hot blast	3	928	·76	585

The mixtures in the above table were made with equal proportions of each iron, unless otherwise stated.

'The bars were all cast, as near as practicable, 1 inch square; those which were found to be defective in this respect were rejected previous to testing. If, however, upon the breaking of the bars, and measuring across the section of fracture, any difference from the true size was discovered, it was noted in the remarks. When the difference was not appreciable, it is stated "rather full in size;" when it was, the dimensions are given as $1\frac{1}{32}$ square, $1\frac{1}{16}$ wide, by $1\frac{1}{32}$ deep, and when this occurs, the breaking weights are reduced to 1 inch square.'—*Appendix to Report of Commissioners appointed to Inquire into the Application of Iron to Railway Structures.* 1849, pp. 390–401.

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* This was the mixture selected for casting the arch ribs of the High Level Bridge.

† White metal, fracture crystalline; very hard and radiating.

TENSILE AND COMPRESSIVE STRENGTH OF VARIOUS DESCRIPTIONS OF IRON.

Description of the iron.	Tensile strength per square inch of section.		Height of specimen.	Crushing strength per square inch of section.		Ratio of the powers to resist tension and compression.	
	lbs.	tons.	inch.	lbs.	tons.		Mean
Low Moor Iron, No. 1 .	12,694 =	5·667	$\frac{3}{4}$	64,534 =	28·809	1 : 5·084	1 : 4·765
			$1\frac{1}{2}$	56,455 =	25·198	1 : 4·446	
Low Moor Iron, No. 2 .	15,458 =	6·901	$\frac{3}{4}$	99,525 =	44·430	1 : 6·438	1 : 6·205
			$1\frac{1}{2}$	92,332 =	41·219	1 : 5·973	
Clyde Iron, No. 1 . .	16,125 =	7·198	$\frac{3}{4}$	92,869 =	41·459	1 : 5·759	1 : 5·631
			$1\frac{1}{2}$	88,741 =	39·616	1 : 5·503	
Clyde Iron, No. 2 . .	17,807 =	7·949	$\frac{3}{4}$	109,992 =	49·103	1 : 6·177	1 : 5·953
			$1\frac{1}{2}$	102,030 =	45·549	1 : 5·729	
Clyde Iron, No. 3 . .	23,468 =	10·477	$\frac{3}{4}$	107,197 =	47·855	1 : 4·568	1 : 4·518
			$1\frac{1}{2}$	104,881 =	46·821	1 : 4·469	
Blaenavon Iron, No. 1 .	13,938 =	6·222	$\frac{3}{4}$	90,860 =	40·562	1 : 6·519	1 : 6·149
			$1\frac{1}{2}$	80,561 =	35·964	1 : 5·780	
Blaenavon Iron, No. 2, first sample .	16,724 =	7·466	$\frac{3}{4}$	117,605 =	52·502	1 : 7·032	1 : 6·577
			$1\frac{1}{2}$	102,408 =	45·717	1 : 6·123	
Blaenavon Iron, No. 2, second sample .	14,291 =	6·380	$\frac{3}{4}$	68,559 =	30·606	1 : 4·797	1 : 4·796
			$1\frac{1}{2}$	68,532 =	30·594	1 : 4·795	
Calder Iron, No. 1 .	13,735 =	6·131	$\frac{3}{4}$	72,193 =	32·229	1 : 5·256	1 : 5·394
			$1\frac{1}{2}$	75,983 =	33·921	1 : 5·532	
Coltness Iron, No. 3 .	15,278 =	6·820	$\frac{3}{4}$	100,180 =	44·723	1 : 6·557	1 : 6·611
			$1\frac{1}{2}$	101,831 =	45·460	1 : 6·665	
Brymbo Iron, No. 1 .	14,426 =	6·440	$\frac{3}{4}$	74,815 =	33·399	1 : 5·186	1 : 5·216
			$1\frac{1}{2}$	75,678 =	33·784	1 : 5·246	
Brymbo Iron, No. 3 .	15,508 =	6·923	$\frac{3}{4}$	76,133 =	33·988	1 : 4·909	1 : 4·936
			$1\frac{1}{2}$	76,958 =	34·356	1 : 4·963	
Bowling Iron, No. 2 .	13,511 =	6·032	$\frac{3}{4}$	76,132 =	33·987	1 : 5·635	1 : 5·555
			$1\frac{1}{2}$	73,984 =	33·028	1 : 5·476	
Ystalyfera Anthracite Iron, No. 2 . .	14,511 =	6·478	$\frac{3}{4}$	99,926 =	44·610	1 : 6·886	1 : 6·735
			$1\frac{1}{2}$	95,559 =	42·660	1 : 6·585	
Yniscedwyn Anthracite Iron, No. 1 . .	13,952 =	6·228	$\frac{3}{4}$	83,509 =	37·281	1 : 5·985	1 : 5·811
			$1\frac{1}{2}$	78,659 =	35·115	1 : 5·638	
Yniscedwyn Anthracite Iron, No. 2 . .	13,348 =	5·959	$\frac{3}{4}$	77,124 =	34·430	1 : 5·778	1 : 5·712
			$1\frac{1}{2}$	75,369 =	33·646	1 : 5·646	
Mr. Morries Stirling's Iron, denominated second quality .	25,764 =	11·502	$\frac{3}{4}$	125,333 =	55·952	1 : 4·865	1 : 4·751
			$1\frac{1}{2}$	119,457 =	53·329	1 : 4·637	
Mr. Morries Stirling's Iron, denominated third quality .	23,461 =	10·474	$\frac{3}{4}$	158,653 =	70·827	1 : 6·762	1 : 6·149
			$1\frac{1}{2}$	129,876 =	57·980	1 : 5·536	

In obtaining the tensile strength, the sectional form of the castings was that of a cross.—*Report of Commissioners, &c.,* p. 101.

EXPERIMENTAL INQUIRY ON THE STRENGTH, ETC., OF CAST-IRON BARS.

General Summary of Results of Experiments on Rectangular Bars of Cast Iron, each Bar being reduced to exactly one inch square.

In the following abstract the transverse strength, which may be taken as a criterion of the value of each iron, is obtained from the mean of the experiments, first on the long bars, 4 feet 6 inches between the supports, and next on those of half the length, or 2 feet 3 inches between supports. All the other values are deduced from the 4 feet 6 inch bars.

No. of Iron in the scale of strength.	Names of Iron.	No. of Experiments on each.	Specific Gravity.	Modulus of Elasticity in lbs. per square inch, or stiffness.	Breaking weight in lbs. of bars 4 ft. 6 in. between supports.	Breaking weight in lbs. of bars 2 ft. 3 in. between supports.	Mean breaking weight in lbs. (S.)	Ultimate deflection of 4 ft. 6 in. bars in parts of an inch.	Power of the 4 ft. 6 in. bars to resist impact.	Colour.	Quality.
1	Ponkey, No. 3, cold blast	4	7·122	17,211,000	567	595	581	1·747	992	Whitish gray .	Hard.
2	Devon, No. 3, hot blast*	2	7·251	22,473,650	537	—	537	1·090	589	White . . .	,,
3	Oldberry, No. 3, hot blast . . .	5	7·500	22,733,400	543	537	530	1·005	549	,, . . .	,,
4	Carron, No. 3, hot blast* . . .	2	7·056	17,873,100	520	524	527	1·365	710	Whitish gray .	,,
5	Eglinton, No. 4, from prepared coke†	6	—	—	515	—	515	1·460	751	Light gray . .	Rather hard.
6	Beaufort, No. 3, hot blast . . .	5	7·069	16,802,000	505	529	517	1·599	807	Dullish gray .	Hard.
7	Butterley	4	7·038	15,379,500	489	515	502	1·815	889	Dark gray . .	Soft.
8	Bute, No. 1, cold blast	4	7·066	15,163,000	495	487	491	1·764	872	Bluish gray .	,,
9	Wind Mill End, No. 2, cold blast .	4	7·071	16,490,000	483	495	489	1·581	765	Dark gray . .	Hard.
10	Old Park, No. 2, cold blast . . .	5	7·049	14,607,000	441	529	485	1·621	718	Gray . . .	Soft.
11	Beaufort, No. 2, hot blast . . .	4	7·108	16,301,000	478	470	474	1·512	729	Dull gray . .	Hard.
12	Low Moor, No. 2, cold blast . . .	4	7·055	14,569,500	462	483	472	1·852	855	Dark gray . .	Soft.
13	Buffery, No. 1, cold blast* . . .	5	7·079	15,381,200	463	—	463	1·550	721	Gray . . .	Rather hard.
14	Brimbo, No. 2, cold blast	5	7·017	14,911,666	466	453	459	1·748	815	Light gray . .	,,
15	Apedale, No. 2, hot blast	3	7·017	14,852,000	457	455	456	1·730	791	,, . .	Stiff.
16	Oldberry, No. 2, cold blast . . .	4	7·059	14,307,500	453	457	455	1·811	822	Dark gray . .	Rather soft.
17	Pentwyn, No. 2	4	7·038	15,193,000	438	473	455	1·484	650	Bluish gray .	Hard.
18	Maesteg, No. 2	5	7·038	13,959,500	453	455	454	1·957	886	Dark gray . .	Rather soft.
19	Muirkirk, No. 1, cold blast* . .	4	7·113	14,003,550	443	464	453	1·734	770	Bright gray .	Fluid.
20	Adelphi, No. 2, cold blast . . .	5	7·080	13,815,500	441	457	449	1·759	777	Light gray . .	Soft.
21	Blania, No. 3, cold blast	5	7·159	14,281,466	433	464	448	1·726	747	Bright gray .	Hard.
22	Devon, No. 3, cold blast	4	7·285	22,907,700	448	—	448	·790	353	Light gray . .	Very hard.

† This iron was melted in the cupola, by coke entirely freed from sulphur, by Mr. C. Calvert's process.

EXPERIMENTAL INQUIRY ON THE STRENGTH, ETC., OF CAST-IRON BARS—*continued.*

No. of Iron in the scale of strength.	Names of Iron.	No. of Experiments on each.	Specific Gravity.	Modulus of Elasticity in lbs. per square inch, or stiffness.	Breaking weight in lbs. of bars 4 ft. 6 in. between supports.	Breaking weight in lbs. of bars 2 ft. 3 in. between supports.	Mean breaking weight in lbs. (S.)	Ultimate deflection of 4 ft. 6 in. bars in parts of an inch.	Power of the 4 ft. 6 in. bars to resist impact.	Colour.	Quality.
23	Gartsherrie, No. 3, hot blast	5	7·017	13,894,000	427	467	447	1·557	998	Light gray	Soft.
24	Eglinton, No. 4, common coke	6	—	—	427	—	427	1·870	798	Dull gray	Rather hard.
25	Frood, No. 2, cold blast	5	7·031	13,112,666	460	434	447	1·825	841	Light gray	Open.
26	Lane end, No. 2	3	7·028	15,787,666	444	—	444	1·414	629	Dark gray	Soft.
27	Carron, No. 3, cold blast	5	7·094	16,246,966	444	443	443	1·336	593	Gray	,,
28	Dundyvan, No. 3, cold blast	4	7·087	16,534,000	456	430	443	1·469	674	Dull gray	Rather soft.
29	Maesteg (marked red)	5	7·038	13,971,500	440	444	442	1·887	830	Bluish gray	Fluid.
30	Corbyns Hall, No. 2	5	7·007	13,845,866	430	454	442	1·687	727	Gray	Soft.
31	Pontypool, No. 2	5	7·080	13,136,500	439	441	440	1·857	816	Dull blue	Rather soft.
32	Walbrook, No. 3	5	6·979	15,394,766	432	449	440	1·443	625	Light gray	Rather hard.
33	Milton, No. 3, hot blast	4	7·051	15,852,500	427	449	438	1·368	585	Gray	,,
34	Buffery, No. 1, hot blast *	3	6·998	13,730,500	436	—	436	1·640	721	Dull gray	Soft.
35	Level, No. 1, hot blast	5	7·080	15,452,500	461	403	432	1·516	699	Light gray	,,
36	Pant, No. 2	5	6·975	15,280,900	408	455	431	1·251	511	,,	Rather hard.
37	Level, No. 2, hot blast	6	7·031	15,241,000	419	439	429	1·358	570	Dull gray	Soft.
38	W. S. S., No. 2	5	7·041	14,953,333	413	446	429	1·339	554	Light gray	,,
39	Eagle Foundry, No 2, hot blast	4	7·038	14,211,000	408	446	427	1·512	618	Bluish gray	,,
40	Elsicar, No. 2, cold blast	4	6·928	12,586,500	446	408	427	2·224	992	Gray	,,
41	Varteg, No. 2, hot blast	4	7·007	15,012,000	422	430	426	1·450	621	,,	Hard.
42	Coltham, No. 1, hot blast	5	7·128	15,510,066	464	385	424	1·532	716	Whitish gray	Rather soft.
43	Carroll, No. 2, cold blast	4	7·069	17,036,000	430	408	419	1·231	530	Gray	Hard.
44	Muirkirk, No. 1, hot blast *	4	6·953	13,294,400	417	—	418	1·570	656	Bluish gray	Soft.
45	Brierley, No. 2	5	7·185	16,156,133	406	—	418	1·222	494	Dark gray	,,
46	Coedtalon, No. 2, hot blast*	4	6·969	14,322,500	409	424	416	1·882	771	Bright gray	,,
47	Coedtalon, No. 2, cold blast*	5	6·955	14,304,000	403	418	413	1·470	600	Gray	,,
48	Monkland, No. 2, hot blast	3	6·916	12,259,500	402	404	403	1·762	709	Bluish gray	,,
49	Ley's Works, No. 1, hot blast	3	6·957	11,539,333	392	—	392	1·890	742	,,	,,
50	Milton, No. 1, hot blast	4	6·976	11,974,500	353	386	369	1·525	532	Gray	Soft and fluid.
51	Plaskynaston, No. 2, hot blast	5	6·916	13,341,633	378	337	357	1·366	517	Light gray	Rather soft.

Explanation of Table (pp. 73, 74).

The irons with asterisks are taken from the experiments on hot and cold blast irons made by Mr. Hodgkinson and myself for the British Association for the Advancement of Science. See Seventh Report, Vol. VI.

The modulus of elasticity was usually taken from the deflection caused by 112 lbs. on the 4 feet 6 inch bars.

RULE.

To find from the above table the breaking weight of rectangular bars generally:—Calling b and d the breadth and depth in inches, and l the distance between the supports in feet, and putting 4·5 for 4 ft. 6 inches, we have $\frac{4{\cdot}5\, b\, d^2 S}{l}$ = transverse breaking weight in pounds. The value of S, being taken from the table above.

For example: What weight would be necessary to break a bar of Low Moor iron, No. 2, 2 inches broad, 3 inches deep, and 6 feet between the supports? According to the rule given above, we have $b=2$ inches, $d^2=9$ inches, $l=6$ feet, $S=472$ from the table. Then $\frac{4{\cdot}5 \times b\, d^2 S}{l} = \frac{4{\cdot}5 \times 2 \times 9 \times 472}{6} = 6{,}372$ lbs.

In conclusion, and as opening up a new field of observation in connection with the strength of cast iron, we shall quote some of the general results from a very extensive series of analyses, made by order of the United States Government. These analyses appear to have been made with extreme care, and the results, so far as they go, are satisfactory, and point to an explanation of some at least of the variations in the resisting powers of this material. We may premise that the guns of the United States Ordnance department are divided into three classes, according to the tests they have stood and the strength of the metal. A large number of specimens having been taken from guns of each class were submitted to analysis by Mr. Campbell Morfit and Mr. J. C. Booth, and gave the following remarkably consistent average results.

	Specific gravity.	Tensile strength.	Total carbon.	Combined carbon.	Allotropic carbon.
First Class Guns .	7·204	28,805	·0384	·0178	·0206
Second Class Guns .	7·154	24,767	·0376	·0146	·0230
Third Class Guns .	7·087	20,148	·0365	·0082	·0283

The different eeffcts produced by the *hot* and *cold* blast are clearly exhibited in the following table, both in reference to chemical composition and to specific gravity and tensile strength.

Blast.	Specific gravity.	Tensile strength.	Total carbon.	Allotropic carbon.	Combined carbon.	Silicium.	Silicium and combined carbon.	Silicium and total carbon.	Slag.	Slag and allotropic carbon.
Hot	7·065	19,640	·0369	·0292	·0076	·0159	·0235	·0528	·00487	·0341
Cold	7·218	29,219	·0407	·0209	·0208	·0059	·0267	·0476	·00124	·0221

It will be observed that, while there is a very great disproportion in the quantities of each *single* ingredient in the hot and cold blast metal, yet there is nearly the same amount of several combined, such as the slag and allotropic carbon, the amount of silicium and combined carbon, or silicium and total carbon. These numbers are significant, for although there is not a great disparity between the amounts of total carbon produced by hot and cold blast, yet the hot blast has evidently driven off a portion of carbon from combination, so that the cold blast contains two and three-fourth times as much combined carbon. The hot blast metal, however, meets with some compensation for this loss of carbon by reducing, by its intense heat, a larger amount of silica, and assuming silicium.

The wide difference in the amounts of slag in the two metals is also remarkable.

The slag and allotropic (graphitic) carbon being of a brittle nature, and not united with the iron, coat the crystalline plates of metal, and diminish their surface of contact; and, consequently, it follows that the tensile strength of the metal must decrease partly in proportion to the increase of slag and allotropic carbon.

From the long series of experiments made on the strength and other properties of iron, as obtained from nearly the whole of the British irons, a general summary of results was deduced, and these results have been given in most of the journals, pocket-books, and manuals, as an epitome of reference to the qualities which at that time (1835) issued from the different furnaces of Great Britain. Since that period, however, other irons, the products of new furnaces and improved processes, have come into use, and have been added progressively to the table. We have now offered some from the Reports of the Commissioners on Railway Structures, which bear more directly upon the original table, as a continuation of the properties of several new irons that have since made their appearance. With only two or three exceptions, there is, however, little or no change taken place in the mechanical condition of the British irons. The Ystalyfera, Redsdale, and Crawshay irons gave indications of great strength, but this increase might have arisen in part from one or more causes, such as the enlargement of the bars in moulding, and the probability of the results not being reduced to *exactly* one inch square.

PART II.

ON WROUGHT-IRON BEAMS FOR SUPPORTING THE FLOORS OF BUILDINGS, AND FOR OTHER PURPOSES.

WROUGHT-IRON beams are of recent origin, and, with few exceptions, they have been sparingly employed in constructions where their superior strength and greater security must have rendered their application of the utmost importance. In the construction of iron ships they have been used in a variety of forms; in bridges intended for the support of heavy weights, such as railway-trains, their introduction has been of immense value; and they are now almost exclusively used for the cross-beams which support the roadways of tubular girder bridges.

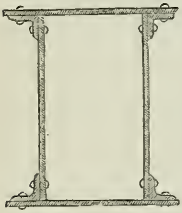

Fig. 32.

At first the box-beam, of which fig. 32 represents a section, was considered superior to the flat beam represented in fig. 33. These two beams have been alternately employed for the purposes above mentioned; but I have invariably given the preference to the plate-beam (fig. 33), on account of its simplicity of construction; and although inferior in strength to the box-beam, it has nevertheless other valuable properties to recommend it.

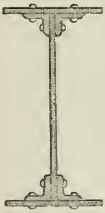

Fig. 33.

On comparing the strengths of these separate beams, weight for weight, it will be found that the box-beam is as 1 : ·93, or nearly as 100 : 90.*

This difference in the resisting powers of the two beams does not arise from any difference or excess in the quantity of material in either structure, but from the better sectional form of the box-beam. The box-beam, it will be observed, contains a larger exterior sectional area, and is consequently stiffer, and better calculated to resist lateral strain, in which direction the plate form generally yields before its other resisting powers of tension and compression can be brought fully into action. Taking this beam, however, in a position similar to that in which it is used for supporting the arches of fire-proof buildings, or the roadway of a bridge where its vertical position is maintained, its strength is very nearly equal to that of the box-beam. But while the plate-beam, in the position thus described, is nearly equal, if not in some respects superior, to the box-beam, t is of more simple construction, less expensive, and more durable, from the circumstance that the vertical plate is thicker than the side-plates of the box-beam, and is consequently better calculated to resist those atmospheric changes which in this climate have so great an influence upon the durability of the metals. Besides, it admits of easy access to all its parts, for purposes of cleaning, painting, &c.

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* See Appendix No. IV.

It is for these reasons that I give the preference to this description of beam; and having had considerable experience in their construction, I am able to state, that they answer exceedingly well for large deck-beams in iron ships, and for any other description of framework in machinery where an irregular or reciprocating motion tends to derange or sever the parts.

From the increased safety and greatly increased strength of the wrought-iron beam, it appears to me to be in every respect adapted for the construction of fire-proof buildings. It offers much greater security, and is free from the risk of those accidents which not unfrequently occur with cast-iron beams, and which have created so much alarm in the public mind. We have already shown the superior adaptation of this material to bridges and other structures where strength and security are the principal objects of its application.* It now becomes a consideration of some importance to exhibit the advantages which may be gained by its introduction, on a large scale, into the building of warehouses, cotton and flax mills, and dwelling-houses, which require protection from risk, whether arising from weakness, from the employment of a dangerous material, or from fire. In these erections it will be found exceedingly valuable, irrespectively of the sense of security which the nature of the material is sure to establish in the public mind. Impressed with these convictions, I unhesitatingly recommend its adoption to the architect and engineer; and provided the laws which govern its strength be carefully attended to, I have every reason to believe that a few examples will not only give entire confidence in its powers, but that increased experience will elicit improved conditions, and probably better forms for its application.* In order the more effectually to guide and encourage the practitioner, I have given a series of drawings exhibiting the principle upon which I would recommend the substitution of wrought iron for the cast-iron beams. I have already stated the objections to cast iron; and in thus directing attention to the introduction of a new material, I have endeavoured to supply the necessary rules and formulæ for computing the strengths, with full and ample details of the construction and other minutiæ connected with the bearings, stay-rods, &c., of these important structures.

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* Since the last edition of this work was published great advances have been made in the use of wrought-iron beams for building purposes. Large factories, hotels, and houses are now built with beams of wrought-iron, and some striking examples of their application will be found when we come to treat of the application of wrought-iron beams to fire-proof buildings.

Another feature in the use of this material is the scope which it gives for an extension of space to any distance commensurate with the convenience of the establishment, or the taste of the architect or engineer. Most of the improved cotton-mills are from 60 to 65 feet in width, with two or three rows of columns, at distances of 15 to 16 feet across the mill, and from 9 to 10 feet in the direction of its length. These columns present serious obstructions to the convenient arrangement and free working of the machinery, but they cannot well be avoided where cast-iron beams are used. By the employment of wrought iron they quickly vanish, as one row of columns in the middle, with only two beams in width, not only meets the objection, but removes all doubts as to the security of the structure. In these constructions, however, it must be borne in mind, that an increase of space is attended with a considerable increase of expense; but when the latter is not a serious consideration, fire-proof mills might be built upwards of 60 feet in width without the introduction of a single column or any other obstruction whatever.

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* Since the above was written, I have successfully introduced this system of construction into a portion of the new fire-proof building recently erected for Messrs. Joseph and James Norton of Wolverhampton. In this building, which is five stories high, several of the arches are supported on wrought-iron beams similar to those represented in fig. 33, p. 78. The arches, as well as the beams of this building, are of great strength, having to support immense quantities of grain and flour, filled at times to the ceiling, exclusively of the vibratory action of the machinery of eighteen pairs of millstones, which are almost always in motion.

In large public buildings this may be effected with perfect ease, and the beams so constructed as to carry a load of 4 to 5 tons to the square yard. Let us, however, return to those erections which require a centre column with a distance of 30 feet between the bearings, as shown in the following woodcut.

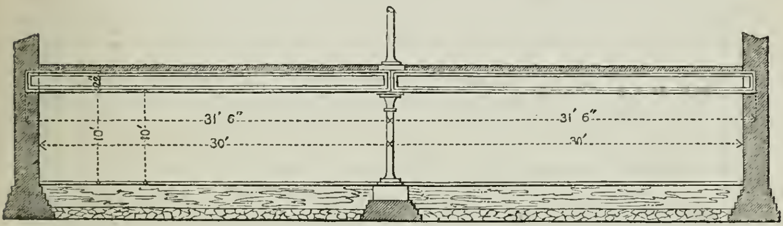

Fig. 34.

In a building of this description, the beams will each be 31 feet 6 inches long, and 30 feet between the supports, and may be composed of plates 22 inches deep, $\frac{5}{16}$ thick, and angle-iron $3 \times 3 \times \frac{1}{2}$ inch at bottom, and $4 \times 4 \times \frac{1}{2}$ at top, riveted on both sides, as shown in the section (fig. 35). The breaking weight of this beam, taking the constant at 75,* would be as follows:—

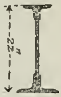

Fig. 35.

Let W represent the breaking weight in tons, a the area of the bottom flange, d the depth of the beam$=$22 inches, and l the distance between the supports$=$360 inches, then we have

$$W = \frac{adc}{l}$$

$= \frac{75 \times 6 \times 22}{360} = 27 \cdot 5$ tons in the middle, or 55 tons distributed equally over the surface. Now, a cast-iron beam of the best form and strongest section, similar to that represented in fig. 31, and calculated to support the same load, would weigh about 2 tons; whereas the wrought-iron beam would only weigh 18 cwt., or less than one-half of the weight of the cast-iron beam. This difference of weight is of considerable importance, as the advantages of using the plate-beams do not consist solely in the saving of more than half of the material, but there is less weight to carry, and much greater certainty as regards the ultimate strength and security of the beams. Let us, however, extend the comparison still further, and endeavour to ascertain the cost of the material and construction of each kind of beam, which, after all, is the only criterion of the utility and fitness of any improvement. Every invention resolves itself into this comparison; and in order to secure a successful application, the superiority of the article (when other things are the same) must be measured by the price at which it can be produced.

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* I have taken 75 as the constant for plate-beams, supported by brick arches or of very large size, instead of 80, used for computing the strength of hollow girders with cellular top. This is done in order to compensate for defects in form which cannot be remedied in the single plate-girder.

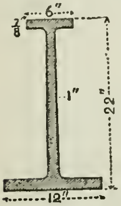

Fig. 36.

Assuming, therefore, that cast-iron beams can be delivered at the foundry at 6*l*. 10*s*. per ton, and that the wrought-iron plate-girders can be manufactured at 16*l*. per ton, it follows that—

A cast-iron beam, 40 cwt., at 6*s*. 6*d*. . . .	£13	0	0
A wrought-iron beam, 18 cwt., 16*s*. . .	14	8	0

making a difference of 1*l*. 8*s*. between the cost of the one and the cost of the other. But, on the other hand, we have, in the case of wrought-iron beams, less than one-half of the weight of metal to carry; and, moreover, the superior lightness of the wrought iron will enable us to erect and fix them in their places at considerably less cost. Altogether, therefore, I am persuaded that the wrought-iron beams, if manufactured on a large scale, can be executed at a moderate rate, and can be made to answer that most desirable object, the combination of strength with lightness, security, and economy. Besides, I am persuaded that beams of this description can be manufactured at 14*l*. per ton instead of 16*l*., as quoted above. If this can be accomplished, there is a direct saving of 8*s*. per ton; a very important economy, when taken in connection with the increased lightness and security.

Should this description of beam become general in its application, it is more than probable that all those under 12 cwt. might be delivered at once, of the required form, from the rolling-mill; and it would be premature to assume, that even the larger sizes, such as we have just described, could not be manufactured in the same way. The skill and intelligence of the iron manufacturers of this country have surmounted greater difficulties; and I have no doubt that a demand has only to be created in order to insure perfect success in all the manipulations connected with that important process. If this could be accomplished, a very important saving of the mineral treasures of the country would be effected; nearly one-half of the metal would be saved, and the price (supposing the beams to be taken from the rolls) reduced to nearly one-half, or from 16*l.* to 8*l.* or 10*l.* per ton. Under these circumstances, cast iron would be no longer admissible for such a purpose, and buildings would be rendered much less liable to the chance of failure, and equally secure from the ravages of fire.

Anticipating these improvements in the manufacture, it is probable that a beam, constructed after the manner described, might take something like the annexed form (fig. 37), the top flange *a* being as much in excess as would equalise the two resisting forces of extension and compression. In every case, however, it would be desirable to retain considerable width in both flanges, in order to give lateral stiffness to the beam, which in wrought iron, owing to the ductile and flexible nature of the material, is a desideratum. When iron is used in its malleable state for constructions of this kind, the cellular top or box-form is evidently one of its most important features, and the strongest to resist compression on the top side. But this cannot be accomplished in the manufacture of beams direct from the rolls without considerable complexity in the construction. An exceedingly strong and simple beam might, however, be constructed with a cellular top, provided that the plate which forms the cell could be rolled upon a mandrel to the required shape, as shown in fig. 38. It would

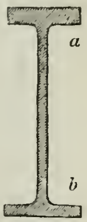

Fig. 37.

be constructed with two cells at *a*, *a*, fixed upon the top edge of the vertical plate, and securely riveted, as at *c*, *c*, from one end of the beam to the other.

This form of beam would probably lessen the difficulties of manufacture, as, instead of double flanges rolled upon the beam, as exhibited in fig. 37, it would only require one at *b*, which would reduce the weight, and afford greater facilities for passing it through the rolls. In the manufacture of the plate which forms the cell some difficulties would, no doubt, have to be encountered; but this, like every other improvement, would yield to perseverance and a determination to succeed.* The object of this form of beam would be a reduction of weight in the top flange. In the cellular construction the top and bottom would be reduced to nearly equal areas, which, in this shape, is the proportion which balances the two resisting forces of extension and compression. In the solid flange it requires nearly double the amount of material on the top to equalise these two forces, or, in other words, to cause the bottom flange to yield to tension at the same time that the top is on the point of giving way to compression. This, however, is a question which must eventually be determined by experiment, and the practical facilities which may be gained in the manufacture.

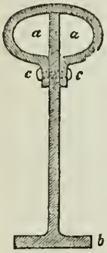

Fig. 38.

We might modify these forms to an almost unlimited extent; but simplicity is so great a desideratum in every mechanical construction, that I am unwilling to multiply the number of designs which readily suggest themselves. Ingenious men are too apt to disregard the consideration that simplicity of form and application very frequently determine the reception or rejection of their inventions, and, as is well known, numerous schemes, full of original thought and admirable talent, have failed from their complexity and over-elaboration of design.

Apprehending some difficulty in the manufacture of the beam represented in fig. 37, I would observe, that unless wrought-iron beams can at once be produced from the rolls similar to that represented in fig. 37, which is evidently the cheapest and best, the next in order in point of cheapness would be that with the bottom flange rolled along with the plate *a*, fig. 39, in the annexed section, and with the top *c* also rolled of ┳ iron, separate in the shape, and riveted along the top edge of the vertical plate, as represented at *d*.

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* In this construction the hood or cellular top would be sufficiently ductile and elastic for the lower side at *c c* to collapse during the process of punching the rivet-holes through both plates at once, and to open again for the reception of the top edge of the plate, to which it is permanently fixed by rivets, as described.

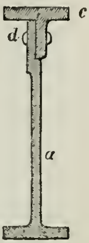

Fig. 39.

The objection to this form of beam would be that the ┳ iron, *c*, embraces only one side of the plate, and is therefore not so convenient, although equally well calculated for resistance to compression, from the circumstance that the middle plate is rolled with a recess to receive the ┳ iron, for the purpose of equalising the forces on each side. In other respects it is a simple construction, and appears to combine the essentials of economy with simplicity of form. Another advantage gained by this construction is, that the mean distance of the rivet-holes in the top part is brought nearer the neutral axis than it is in the box form of beams.

Since the above was written for the first edition of this work, we have had an opportunity of inspecting, at the Paris Exposition of 1855, some rolled beams superior to any hitherto manufactured in this country, and of a size which proves that the difficulty to which we have alluded is not insurmountable. The following figures (40 and 41) will show the sectional dimensions of these beams; they are such as are employed in France for the floors of fire-proof buildings, and appear to answer every purpose in the formation of floors entirely composed of iron joists, plaster of Paris, and hollow bricks. The smaller beam was rolled to a length of 60 feet, and the larger to a length of 40 feet.

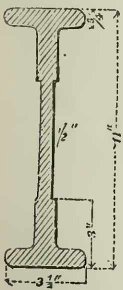

Fig. 40.

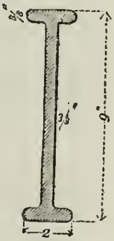

Fig. 41.

These beams, it must be observed, were sent to the Exhibition as specimens of manufacture, and were probably rolled for the purpose; they, however, clearly show that beams of this description can be made direct from the rolls, and we have yet to attain in this country a higher state of perfection in the manufacture, and a more extended application of wrought-iron beams to the construction of dwelling-houses.*

Hitherto we have treated of beams of light weight and short span; instances, however, occur where they are required of

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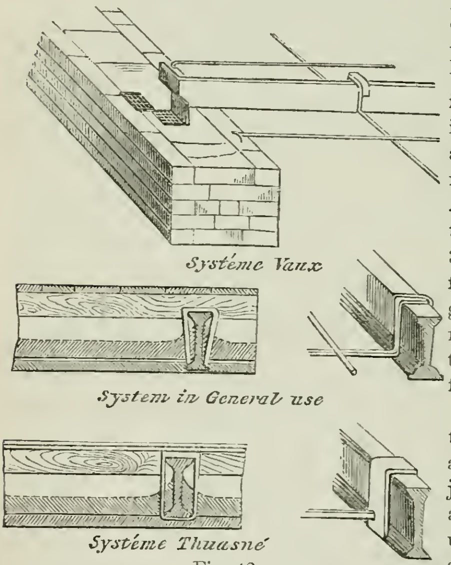
* A system of fire-proof flooring has been in use for some time on the Continent, and indeed has been partially employed in this country. In France two principal systems have been introduced, called respectively the *Système Vaux* and *Système Thuasné*, from the names of their inventors, of which the following sketches will give a better idea than any lengthened description:—In the Système Vaux it will be seen that the beams for supporting the flooring consist of simple plates of wrought iron, split and bent at the end to obtain a firm holding in the wall. These are bound together by tie-rods, which are crossed by other rods supporting the ceiling. In the Système Thuasné wrought-iron flanged joists have been substituted for the plates, and a different method of attaching the tie-rods is employed. The beams generally used are similar to those shown in figs. 40 and 41; and they vary in depth, thickness, and length according to the width of the room and the length of the span. At first they were placed at distances of 1 metre apart = 3 feet $3\frac{1}{2}$ inches; but that distance was found to be inconvenient, not giving sufficient strength and rigidity to the floor; and hence they are now placed at about 2 feet asunder.


Système Vaux

System in General use

Système Thuasné

Fig. 42.

The usual manner of forming the ceiling is to force upwards against the bottom of the iron joists flat boards, which answer as a centering, and then to fill up the spaces between the joists and tie-rods, to a depth of $2\frac{1}{2}$ or 3 inches, with a coarse grout of plaster of Paris. This hardens almost immediately, and forms a ceiling ready to receive the finishing coat of fine plaster. The upper part above the iron joist is then filled up with hollow brick, or small cylinders of baked clay, like flower-pots; these, being again grouted, form an excellent bond to the iron joists. On the top of these groutings may be formed the floor of tiles or concrete, as most convenient; or a wooden floor may be introduced, which is frequently done, by embedding wooden sleepers at the required distances to receive the flooring.

M. Thuasné published, some years since, a table of sizes of joists and prices for large span and considerable strength, and in recommending this peculiar application, it may be necessary that we should meet these requirements by the introduction of a construction suitable for such purposes. To accomplish these objects, the beams just described are not exactly those we should recommend; but assuming that the smaller description can be rolled to the required form, we have yet to provide for those which require a span from thirty up to fifty feet. In public buildings and in the cross-beams for bridges supporting roadways,

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the use of builders and the public, of which Mr. Burnell gave the following translation in a paper read to the Royal Institute of British Architects in 1854:—

Bearings.					Depth of joist in inches.	Depth of floor complete in inches.	Weight per square in lbs.	Iron work per square.			Including grouting (12*s.*) per square.		
Ft.	In.		Ft.	In.				£	*s.*	*d.*	£	*s.*	*d.*
10	0	to	11	6	4	$7\frac{1}{8}$	370	2	19	5	3	11	5
11	6	,,	13	0	$4\frac{3}{4}$	$7\frac{7}{8}$	420	3	6	5	3	18	5
13	0	,,	16	6	$5\frac{1}{2}$	$8\frac{5}{8}$	465	3	14	4	4	6	4
16	6	,,	20	0	$6\frac{1}{4}$	$9\frac{1}{2}$	510	4	1	9	4	13	9
20	0	,,	23	0	$7\frac{1}{8}$	$10\frac{1}{4}$	605	4	17	6	5	9	6
23	0	,,	26	0	$8\frac{5}{8}$	$11\frac{3}{4}$	700	5	12	4	6	4	4

Floors in many respects similar to these have been introduced into England by Messrs. Fox & Barrett.

Sometimes the French floors are constructed in a different manner: the joists being laid as before, cross tie-rods are placed at about every 3 feet 6 inches, and on these slender wrought-iron rods rest, three between each joist. These rods are run through perforated bricks built in a slightly arched manner, the space below them being filled up with plaster of Paris, as shown in fig. 43; over the joists wooden sleepers are placed to receive the boarding for the floors.

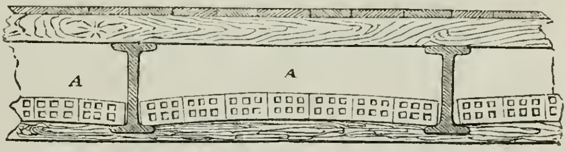


Fig. 43.

In this description of floor there is every security from fire; and the plaster being a bad conductor of heat, equalises the temperature of the room. The only objection is the open space A A, between the arches and the boarding, serving as a receptacle for vermin; but this objection may be removed by divisions of plaster 6 inches thick, carried across the floor and in contact with the boards. This description of building is in general use in Paris and other towns of France; and viewing it as a permanent fire-proof structure, I should earnestly recommend its adoption in this country.

thoroughfares, &c., we have frequent examples where lightness united to strength becomes an important element in the construction, and it will at once become apparent that provision for these and similar structures should be made, in order to afford the necessary facilities for their adaptation to structures of that description.

We have already remarked that the smaller description of wrought-iron beams may be produced at once from the rolling-mill at a very moderate price per ton, and there being a direct saving of more than one-half in weight, the actual cost will be considerably less than that of cast-iron beams of equal strength. In cases where the extent of span required would render it impracticable to roll the beam in one piece, convenient weights might be rolled into sections of the proper form, and a beam of an excellent description be constructed by joining the parts together, as shown in the following sections and elevation, figs. 44, 45, and 46:—

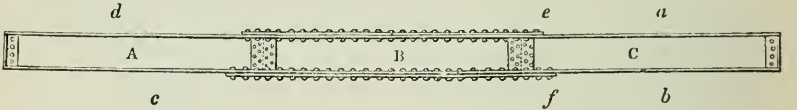


Fig. 44.

In this construction the parts A, B, and C are rolled in three lengths to the form, as shown in fig. 45, which is a section through the line *a b*, and being united by proper covering plates and rivets, it will form a section at the junction through the lines *c d* and *e f*, as exhibited in fig. 46. This construction may be carried to a span of forty to fifty feet; and provided the covering plates are properly proportioned and the riveting well executed, the beam will be equal in strength to one formed of solid iron without the intervention of a single joint.

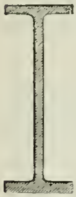


Fig. 45.

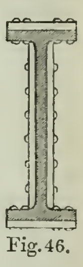


Fig. 46.

I have already stated that one great advantage of this construction consists in the absence of rivet-holes; in order to show this advantage more definitely, let us suppose that there are four rivet-holes at the bottom of the beam, and that the section of each is a quarter of an inch, then it follows that a beam without rivet-holes, whose bottom section is $6 - \frac{3}{4}$ inches, or $5\frac{1}{4}$ inches, will bear as much as a beam with the rivet-holes whose bottom section is 6 inches; thereby showing that there would be a great economy of material in the use of beams rolled in the manner described.

It is probable that the rectangular box-beam is more appropriate for the support of great weights on a large span, than the plate-beam recommended above; but I have already stated my objections to the box-beam (p. 79), and the same reasoning will apply to it in this case, viz. the danger of oxidation and the impossibility of reaching the interior for the purposes of painting, cleaning, &c. These are the chief considerations which induce me to give the preference to the flat beam; and I am of opinion that, with proper care in the construction, in spans up to forty, and in some cases fifty feet, they will be found superior to any other description of beam. In cases where the distance between the supports exceeds fifty feet, the tubular girder is evidently the best form of beam; and this we shall fully enter into when we come to treat of the subject of bridges.

In the following section will be found the experimental data which have led me to these views, and induced this strong recommendation of a simple rolled wrought-iron beam or joist in the construction of buildings.

EXPERIMENTS ON THE STRENGTH, ETC., OF WROUGHT-IRON BEAMS.

In addition to the experiments which have already been recorded, it will be instructive to refer to the series made in 1845, to determine the strength and form of the Conway and Britannia Tubular Bridges.*

In reference to these experiments in particular, it has been observed that—‘They are not only highly interesting in themselves, but involve practical considerations of deep importance in relation to the future interests of civilised society. They show that the rectangular form of the tubular wrought-iron girder is much better calculated to resist transverse strain than any other form that can be adopted, provided that the sectional parts are so arranged and distributed as to give the greatest strength with the least quantity of material.'

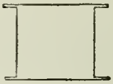
---

* For a more enlarged view of these experiments, see my work on the Conway and Britannia Tubular Bridges.

Experiments on the Transverse Strength of Malleable Iron Rectangular Tubes.

Experiment XIV., *July* 31, 1845.—Rectangular tube 18 feet 6 inches long, 9·6 square, and 17 feet 6 inches between the supports.

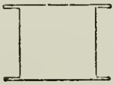
Thickness of plates, top . . $\frac{1·05}{14} = ·075$ inch.
" " bottom $\frac{1·04}{14} = ·0743$ "
" " sides . $\frac{1·04}{14} = ·0743$ "
Weight of tube = 202 lbs.
Weight of shackle = 938 lbs.

Weight in lbs.	Deflection in inches.	Deflection, load removed.	Remarks.
938	·17		
2,058	·55		
3,178	·96		
3,738	—	—	Yielded to compression, the top side doubling up, and the sides bulging close to the injured part.
∴ Ult. deflection = 1·12			

The great weakness indicated by this tube in yielding to a weight of only 3,738 lbs. caused a different arrangement of the plates. A new one, of nearly three times the thickness, was prepared and riveted on the top side: the plate on the lower side was also strengthened at the joints; and having repaired the other parts injured, the tube was again put to the test, as follows:

Experiment XIV.*a*, *October* 9, 1845.—Rectangular tube 18 feet 6 inches long, 9·6 inches square, and 17 feet 6 inches between the supports.

Thickness of plates, top . . $\frac{1·26}{5} = ·252$ inch.
" " bottom $\frac{1·05}{14} = ·075$ "
" " sides . $\frac{1·04}{14} = ·074$ "
Weight of tube = 384 lbs.
Weight of shackle = 960 lbs.

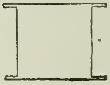
Weight in lbs.	Deflection in inches.	Deflection, load removed.	Remarks.
960	·07		
1,854	·16		
2,717	·25		
3,568	·35		
4,441	·45	·07	
5,310	·55	·10	
6,180	·67	·11	
6,180	·73	—	This weight was repeated, in consequence of the tube being slightly distorted.
7,017	·84	·14	
7,427	·94	·20	
7,859	1·07	·29	
8,273	—	—	Broke asunder by tearing at a joint on the bottom side 11 inches from the shackle, where the plate was weakened.
∴ Ult. deflection = 1·10			

Here, by increasing the material at the top of the tube, we attained more than double the strength,—thereby showing that fibrous bodies like wrought iron, being more ductile, are more susceptible of injury from compression than from extension.

This law is further confirmed by the succeeding experiments.

Experiment XV., *July* 31, 1845.—Rectangular tube 18 feet 6 inches long, 9·6 inches square, and 17 feet 6 inches between the supports.

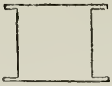
Thickness of plates, top . . $\frac{1·06}{14} = ·0757$ inch.
„ „ bottom $\frac{1·14}{8} = ·1425$ „
„ „ sides . $\frac{1·06}{14} = ·0757$ „
Weight of tube = 255 lbs.
Weight of shackle = 988 lbs.

Weight in lbs.	Deflection in inches.	Deflection, load removed.	Remarks.
988	·16	—	In this experiment great weakness is exhibited, as well as in the former one.
2,108	·45	·05	
3,228	·80	·09	
3,788	—	—	With this weight the top plate began to buckle 2 feet 6 inches from the shackle on one side, and 6 inches from it on the other. It appears to require stiffness in order to resist the tendency to pucker.
∴ Ult. deflection = ·94.			

This experiment was repeated with a strong plate 2 feet 7 inches long, 11 inches wide, and ·11 inch thick, laid along the top, in order to stiffen it and throw the strain more upon the bottom plate. The results were, however, unimportant, until the tube was reversed, with the thick side upwards, when a very important change was effected, as shown in the succeeding experiment.

Experiment XV.*a*, *July* 31, 1845.—Rectangular tube the same as before; tube reversed with the thick side uppermost.

Thickness of plates, top . . $\frac{1·14}{8} = ·142$ inch.
„ „ bottom $\frac{1·06}{14} = ·0757$ „
„ „ sides . $\frac{1·06}{14} = ·0757$ „
Weight of tube = 255 lbs.
Weight of shackle = 988 lbs.

Weight in lbs.	Deflection in inches.	Deflection, load removed.	Remarks.
988	·17	—	The deflection, as well as the modulus of elasticity, are much greater in this than in any of the former experiments.
2,108	·50	·07	
3,228	·78	·14	
3,788	·90	·18	
4,348	1·05	·20	
4,908	1·21	·26	
5,468	1·37	·32	
6,028	1·54	·40	
6,588	1·75	·50	
7,148			
∴ Ult. deflection = 1·76			

If we compare the results of the last two experiments with those contained in Experiments XIV. and XV.*a*, we shall find that the proportions of the top and sides are widely different. In both cases, however, when the tube was reversed, with the thick side uppermost, double, or nearly double, the strength is obtained. Hence it follows that, in order to obtain the section of greatest strength, the top side of a tube, when submitted to a transverse strain, must be considerably thicker than its lower side.

This fact is fully established in every succeeding experiment, as well as in those already recorded, for the tube almost constantly gave way to compression, unless secured by stronger plates on the top side.

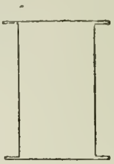
EXPERIMENT XVI., *August* 1, 1845.—Rectangular tube 18 feet 6 inches long, 18·25 inches deep, 9·29 wide,and 17 feet 6 inches between the supports.

Thickness of plates, top . . $\frac{1\cdot340}{9} = \cdot1490$ inch.

" " bottom $\frac{1\cdot345}{5} = \cdot2690$ "

" " sides . $\frac{0\cdot950}{16} = .0594$ "

Weight of tube = 317 lbs.
Weight of shackle = 988 lbs.

Weight in lbs.	Deflection in inches.	Deflection, load removed.	Remarks.
988	·15		
2,108	·30		
3,228	·44	·05	
4,348	·60	·07	
5,468	·70	·10	
6,588	1·00	·22	
6,812	—	—	With this weight the top plate began to rise 18 inches from the shackle, after sustaining the weight for about a minute.
∴ Ult. deflection = 1·03			

Having, in this experiment, crippled the upper side of the tube, it was turned upside down after the injured part was straightened, and the experiment repeated.

In most of the experiments the tendency to rupture was slow and progressive—a property which seems to be inherent in sheet iron tubes, particularly when they yield to compression.

Under this species of strain, destruction is never instantaneous, as in cast iron, but advances gradually, the material emitting during the process a crackling noise for some time before the experiment is complete and absolute rupture takes place.

EXPERIMENT XVI.*a*, *August* 1, 1845.—The preceding tube reversed, with the thick side uppermost.

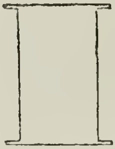
Thickness of plates, top . . $\frac{1·345}{5} = ·2690$ inch.

„ „ bottom $\frac{1·340}{9} = ·1490$ „

„ „ sides . $\frac{0·950}{16} = ·0594$ „

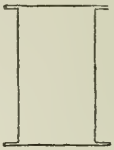
Weight of tube = 317 lbs.

Weight of shackle = 988 lbs.

Weight in lbs.	Deflection in inches.	Deflection, load removed.	Remarks.
988	·80		
2,108	·30		
3,228	·42	·03	
4,348	·55	·10	
5,468	·70	·15	
6,588	·80	·20	
7,708	·92	·24	
8,828	1·10	·30	
9,948	1·30	·31	* *August* 2. The weight, 11,068 lbs., was left on the tube from half-past 3 o'clock P.M. till the following day at half-past 9 A.M., when the deflection increased from 1·45 to 1·60.
10,508	1·35	·32	
11,068*	1·40	·40	
11,068†	1·60	·50	† Experiment continued after sustaining the weight 18 hours.
11,628	1·65	·53	
12,188	—	—	With this weight the top side puckered.
∴ Ult. deflection = 1·73			

The tube failed with 12,188 lbs. at two of the joints on the top side, 3 feet from the shackles. This failure was accompanied by the sides bending inwards on one side, with a similar indentation on the other, and the top plate doubling at the joints, in the form of the letter S. The breaking weight is nearly twice as much as with the thick flange below.

Experiment XVI.*b*, *September* 20, 1845.—The top side still yielding to compression, a stronger plate was riveted upon it; and in order to cause the bottom to give way to a tensile strain, a thicker plate was riveted over the joint on the bottom side, and the experiment repeated. Distance between the supports as before, 17 feet 6 inches; weight of shackle = 960 lbs.

Weight in lbs.	Deflection in inches.	Deflection, load removed.	Remarks.
960			
2,697	·09		
4,426	·16		
6,173	·25		
7,859	·34		
9,555	·42		
11,262	·60	·07	
12,107	·65	·14	
12,990	·72	·15	
13,867	—	—	Broke after sustaining the weight some minutes, by tearing the rivets from the joints on the upper side, 3 feet 8 inches from the shackle.
∴ Ult. deflection = ·76			

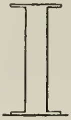
The great powers exhibited in the last experiment by the addition of a certain quantity of material to the upper side of the tube suggested a further extension of the experiments, with some slight modifications of form in order to render more conclusive the principle which the previous trial had indicated.

For this purpose a hollow girder 25 feet long and 15 inches deep, of the following dimensions, was constructed and submitted to experiment.

Experiment XVII., *August* 2, 1845.—Rectangular tube or girder 25 feet $1\frac{1}{4}$ inches long, 15 inches deep, $2\frac{1}{4}$ inches wide, and 24 feet between the supports.

Thickness of plates, top . . $\frac{1\cdot300}{5}=\cdot260$ inch.
„ „ bottom $\frac{1\cdot300}{5}=\cdot260$ „
„ „ sides . $\frac{1\cdot180}{9}=\cdot131$ „

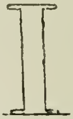
Weight of tube = 788 lbs.
Weight of shackle = 800 lbs.

Weights in lbs.	Deflection in inches.	Deflection, load removed.	Remarks.
800	·07	—	The elasticity remains nearly perfect up to 8,640 lbs.
1,920	·20		
3,040	·33		
4,160	·50		
5,280	·60		
6,400	·70		
7,520	·83		
8,640	·95	·07	
9,760	1·20	·15	
10,880	1·35	·20	
12,000	1·50	·25	
13,120	—	—	Broke by tearing through the solid plate on the bottom side, 7 inches from the shackle, as the weight was laid on.
∴ Ult. deflection = 1·613			

A flaw having been discovered in the plate where the fracture took place from imperfect welding, a stronger plate, 14 in. long and one-fourth of an inch thick, was riveted over the crack, and the experiment repeated.

EXPERIMENT XVII.*a*, *August* 4, 1845.—Rectangular tube the same as before.

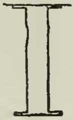
Dimensions
- Top flange . . 6 inches × 0·260 inches.
- Bottom flange . 10 „ × 0·260 „
- Sides . . . . 15 „ × 0·31 „
- Between sides . . . . . 2·25 „

Weight in lbs.	Deflection in inches.	Deflection, load removed.	Remarks.
5,280	·65	·08	
6,460	·77	·15	
7,520	·90	·18	
8,640	1·05	·23	
9,760	1·20	·30	
10,880	1·31	·21	
12,000	1·46	·21	The defects of elasticity were not, from some unknown causes, clearly indicated, until the weight 14,240 lbs. was removed, when the defects were found = ·60. Probably this might have arisen from some unequal tension.
13,120	1·60	·21	
14,240	1·75	·60	
14,800	2·11	·62	
15,360	2·17	·62	
15,920	2·28	·68	
16,480	2·36	·74	
17,040	2·38	·80	
17,600	—	—	With this weight the top plate gave way by compression.
∴ Ult. deflection = 2·66			

As this description of beam indicated very considerable powers of resistance, it was deemed advisable still further to test its powers by allowing the weight 14,240 lbs. to remain suspended during the night. This was done during a period of seventeen hours, after which the load was removed. The deflection during that time had increased from 1·75 to 2·00 = ·25, and the loss in elasticity was ·60 — ·30 = ·3.

The beam during the last two experiments had suffered considerably from the severity of the strains to which it had been subjected; and it was considered that the anomalous condition of puckering, which had all along been present, might be avoided by reversing the girder with the broad flange uppermost. This was accordingly done, the injured part having first been straightened, and a strong plate 19 inches long having been riveted upon it, the experiment was again proceeded with, as follows:—

Experiment XVII.*b*, *August* 5, 1845.—Rectangular tube, same as before; the beam reversed, with the narrow flange downwards.

Weight in lbs.	Deflection in inches.	Deflection, load removed.	Remarks.
9,760	1·40	·38	The deflection and permanent sets must be added to and subtracted from the respective numbers 1·40 and ·38.
10,880	1·65	·50	
12,000	1·83	·59	
13,120	2·03	·69	
14,240	2·30	·84	
15,360	2·49	·97	
15,920	—	—	Broke when the weight was laid on by extension, the lower plate tearing asunder 6 inches from the centre of the shackle.
∴ Ult. deflection = 2·58			

Owing to the broad flange being placed uppermost, it was expected that the tube would yield to extension, which was the case; but the plate gave way at the rivets of a joint at some distance from the centre. This joint, moreover, had been a good deal strained by the former experiments, which may account for its fracture by a comparatively less weight.

Experiment XXV., *September* 20, 1845.—Having tested the powers of the larger description of girder in a variety of ways, the smaller one was treated in the same manner, as follows:—

Rectangular girder 12 feet long, 8 inches deep, 1 inch wide, and 11 feet between the supports.

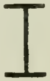
Thickness of plates, top . . $\frac{1\cdot41}{5} = \cdot282$ inch.

„ „ bottom $\frac{1\cdot16}{10} = \cdot116$ „

„ „ sides . $\frac{1\cdot01}{15} = \cdot067$ „

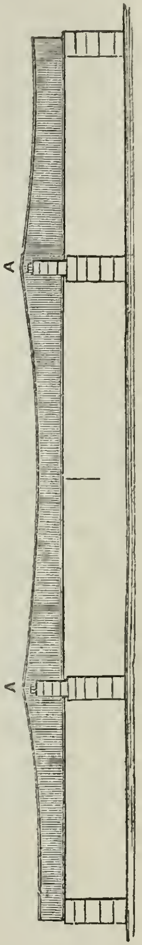
Weight of tube = 125 lbs.

Weight of shackle = 930 lbs.

Weight in lbs.	Deflection in inches.	Deflection, load removed.	Remarks.
930	·06	—	When 9,974 lbs. were laid on, the ends, from the extreme thinness of the plates, were giving way at the supports; two pieces of hard wood were, however, inserted between the sides, which kept them straight and prevented them from twisting.
1,780	·11	—	
2,630	·16	—	
3,516	·21	—	
4,382	·26	—	
5,214	·32	—	
6,105	·37	·010	
6,543	·41	·012	
6,996	·44	·035	
7,433	·47	·040	
7,861	·51	·050	
8,273	·54	·058	
8,693	·58	·075	
9,107	·62	·097	
9,545	·67	·118	
9,974	·74	·150	
10,386	·87	·243	The top plate doubled up with this weight 4 inches from the shackle. Finding the top of the beam the weakest, it was turned upside down, and a weight of 6,113 lbs. laid on to straighten the injured part.
10,827	1·06	·325	
11,254	—	—	
Beam reversed.			
6,113	·51	·10	
6,549	·60	·13	
6,978	·74	·23	
7,146	—	—	With this weight the top plate was forced upwards.
∴ Ult. deflection = ·75			

This girder, although extremely light on the sides, with a tolerably thick top, nevertheless gave way by compression. Its bearing powers were very considerable with the thick side uppermost; and provided that part had contained a little more material, it would have carried upwards of 12,500 lbs. During the progress of the experiment I had frequent conferences with Mr. Stephenson; and having reported to him from time to time the results that were obtained, and the impression they made upon my mind, he suggested that it might be desirable to have a tube made of an entirely different form, in order, if possible, to throw the top side as well as the bottom of the tube into a state of tension. This suggestion was intended to obviate the anomalous condition of puckering, and to prevent as much as possible that tendency to 'buckle,' which, in every instance, is more or less present in rolled sheet-iron plates.* It had a further object in view; namely, to relieve the strain on the centre of the tube, whether arising from the effects of its own weight of the load, by extending the length beyond the supports to a distance of half the span on each side. This additional weight, extending over the piers, was expected to act as a counterpoise; A A being the fulcrum to that part of the tube in the middle; and it would also assist in the support of the load during the passage of the trains through the tubes. For these objects a tube was made of the form shown in fig. 47.

Fig. 47.



Experiment XVIII.*a*, *August* 3, 1845.—Rectangular tube 37 feet 8 inches long, 13·25 inches deep in the middle, 7½ inches wide, with the upper part raised to 17·25 inches, as at A A, and 18 feet between the supports. The width of the top and bottom plates was as follows:—

Thickness of plates, top $\frac{1\cdot140}{8} = \cdot1425$ inches.

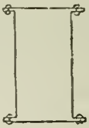
" " bottom $\frac{1\cdot140}{8} = \cdot1425$ "

" " sides $\frac{1\cdot240}{11} = \cdot1127$ "

Weight of tube = 640 lbs.
Weight of shackle = 800 lbs.

---

* It is almost next to impossible to roll plates with all the parts in the same degree of tension. Almost every plate has more or less buckle, and it requires no inconsiderable degree of skill to elongate those parts of a plate where the tension is greatest, and also to find out the parts which cause the 'buckle.' A considerable portion of variable tension in the composition of plates is probably caused by unequal contraction in the process of cooling, and also by a difference of temperature in the blooms from which the plates are rolled.

Weight in lbs.	Deflection in inches.	Deflection, load removed.	Remarks.
800	·09	—	
1,920	·20	·02	
3,040	·32	·05	
4,160	·45	·09	
5,280	·59	·16	
6,400	·71	·19	
7,520	·84	·22	
8,640	·99	·27	
9,760	1·18	·32	
10,880	—	—	With this weight the top plate doubled up 1 foot 6 inches from the shackle.
∴ Ult. deflection = 1·31			

After the top side had yielded to compression the weights were removed, and the supports under the bottom having been lowered, the tube was supported on two cross-bars passing through the tube, as at A A, fig. 47, and the weights were again suspended upon it.

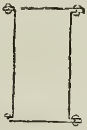
These weights, when laid on, made no difference in changing the direction of the forces, as the top plate was again forced upwards by compression, and that to a greater height than before, accompanied with increased distortion of the sides, which shortly became collapsed diagonally on each side of the shackle. In this experiment it is probable that some degree of tension was induced along the upper line of the top plate, as the extreme end of the tube was raised with some force as the weights were increased. This property of the weight raising the end over the fulcrum A A was strikingly apparent when the whole weight = 10,800 lbs. was laid on; but it did not appear to alter the conditions of the middle part, which was forced upwards by compression, and followed the same law as if it had been formed of a single beam.

These appearances indicated a tendency of the two ends, then extended to half the distance between the supports, to act as a counterpoise, and not only to change the direction of the strain on the top side, but relieve the bottom, which in other respects must have borne the whole weight. From this it is inferred that the tube in its full size would be greatly relieved by increasing its length on each side of the land-towers, as in the case of the Britannia Bridge, to the extent of half the span.

As a further illustration of these views, the injured part of the tube was repaired by riveting an additional plate of the same thickness along the top side over that previously damaged. With this addition the experiment was again repeated.

Experiment XVIII. repeated, *August* 10, 1845.—Rectangular tube same as before.

Thickness of plates, top . . . = ·2850 inches.
„ „ bottom . = ·1425 „
„ „ sides . . = ·1127 „
Distance between the supports, 18 feet.

Weight in lbs.	Deflection in inches.	Deflection, load removed.	Remarks.
7,520	·68		
8,640	·84	·04	
9,760	·99	·10	
10,880	1·15	·16	
12,000	1·31	·25	With this weight, 12,000 lbs., the sides were slightly puckered, indicating a tendency to force the top side upwards.
12,560	1·40	·32	
13,120	1·64	·42	
13,680	—	—	Puckered as before by compression, the top plate doubling up 13 inches from the shackle.
∴ Ult. deflection = 1·71			

On consulting the two last experiments, it is obvious that no great increase of strength was obtained by doubling the thickness of the top plate. This may, however, be accounted for by the circumstance of the top plates being under instead of above the line of compression.

In every description of girder composed of malleable iron, and probably of any other material, the upper side should be elevated to a greater height than the line of ultimate deflection. It should always be above, but never below, the horizontal line of compression. Another cause of the failure of this tube, with a comparatively less weight than the increased thickness of the top would indicate, might be traced to the severe injuries which that part sustained in the previous tests. Hence followed the puckering of the top side, at a much earlier stage of the experiment than it should have done had the plates been sound and the line of the forces changed. The experiments on this form of tube are perhaps the more interesting from the fact that they exhibit certain defects of form which it may be desirable to avoid in girders of this description. If the parts suspended beyond the piers are intended to act as a counterpoise to the load, it will then become necessary to have the girder of uniform strength and texture, with a slight curvature of the top side about one-tenth the depth. With these precautions in the construction, the strength would be greatly improved, and, being subjected to severe strain, will follow the same law, as regards extension and compression, as those of a girder of the simple form.

During the progress of Experiment XXII., when the elliptical tube, after being strengthened by an iron cellular fin, riveted along the top, was found defective in resisting the crushing force, it then occurred that a different construction might be introduced, so as to give strength and rigidity to that part. For this purpose I sketched out, and gave orders for the construction of, a tube with a corrugated top, forming two longitudinal cavities along the whole of its top side, as exhibited in the annexed sketch, Experiment XXIX.

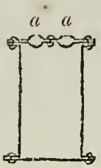
This tube was executed with considerable care; and having been submitted to the usual experimental test, the results were as follow:—

EXPERIMENT XXIX., *October* 14, 1845.—Rectangular tube, with a corrugated top 19 feet 8 inches long, 15·4 inches deep, 7·75 inches wide, and 19 feet between the supports.

Thickness of plates, top . . $\frac{\cdot 23}{2}$ = ·115 inches each.
„ „ bottom = ·180 „
„ „ sides . = ·070 „

Weight of tube = 500 lbs.
Weight of shackle = 988 lbs.

The tubes or cells *a a* were 1·65 inches in diameter.

Weight in lbs.	Deflection in inches.	Deflection, load removed.	Remarks.
988	·035		
2,736	·110		
4,468	·190		
6,215	·270		
7,924	·340	·020	
9,636	·424	·050	
11,334	·523	·062	
13,041	·640	·095	
14,751	·735	·125	
16,490	·870	·186	
18,205	1·070	·276	With the weight 18,205 lbs. the deflection increased ·02 inches in three minutes.
19,065	1·155		
19,918	1·270	·400	
20,764	1·425		
21,629	1·520	·590	
22,469	—	—	Broke by the side plate tearing from the top at 2 feet from the shackle.
∴ Ult. deflection = 1·59			

A short time previous to the tearing of the sides from the top at the rivets, that part had begun to assume a slightly undulating appearance on one side, arising from the weakness of the side plate, which gave way near to the shackle. This was not, however, the only part that suffered under the strain, as the opposite side was tearing from the bottom plate, at the same time evidently showing a rapid approach to rupture on both the lower and upper sides of the tube. These parts exhibited important features in the due and perfect adjustment of the top and bottom, which in this case were calculated to resist, as nearly as possible, the forces acting upon them.

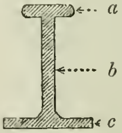
Another property of considerable importance in this description of girder is its progressive tendency downwards to destruction. It is widely different from cast iron and other crystalline substances in this respect, since, from its fibrous nature and greater ductility, it gives timely warning before rupture takes place.

This property was noticed in several of the former experiments; and in this experiment it became more apparent after the whole weight, 22,469 lbs., was laid on.

With this weight more than three minutes elapsed before the experiment was completed, and the tube rendered unfit for use.

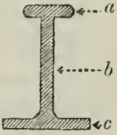
EXPERIMENT XXX., *October* 10, 1845, on a malleable iron beam of the annexed sectional form, 11 feet 7 inches long, and 11 feet between the supports.

Dimensions at $a = 1{\cdot}000$ inches $\times 2\frac{1}{2}$ inches.
„ „ $b = {\cdot}325$ „ $\times 7$ „
„ „ $c = {\cdot}380$ „ $\times 4$ „
Weight of beam = 227 lbs.
Weight of shackle = 885 lbs.

Weight in lbs.	Deflection in inches.	Deflection, load removed.	Remarks.
885	·04		
2,581	·12		
4,317	·20		
6,050	·26		
7,743	·35		
9,493	·46		
11,253	·60	·09	
12,955	—	—	With this weight the beam became distorted, and continuing the weight for some time, the deflection kept increasing until it bent laterally, so as to be no longer able to sustain the load.
∴ Ult. deflection = ·69			

EXPERIMENT XXXI., *October* 10, 1845, on a malleable iron beam of the annexed sectional form, 10 feet 8 inches long, and 10 feet between the supports.

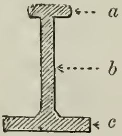
Dimensions at $a = 1{\cdot}000$ inches $\times 2\frac{3}{4}$ inches.
„ „ $b = {\cdot}350$ „ $\times 8$ „
„ „ $c = {\cdot}440$ „ $\times 4{\cdot}30$ „
Weight of beam = 247 lbs.
Weight of shackle = 885 lbs.

Weight in lbs.	Deflection in inches.	Deflection, load removed.	Remarks.
885			
2,631	·04		
4,358	·12		
6,098	·15		
7,827	·19		
9,585	·21		
11,278	·26		
12,980	·30	·03	
14,693	·35	·03	
16,373	·45	·09	
18,115	·68	·26	
18,962	—	—	With this weight the beam was distorted, and the experiment discontinued.
∴ Ult. deflection = ·71			

In both these experiments the beams yielded to lateral deflection, showing certain defects of form, arising from want of lateral strength and breadth in the top and bottom flanges.

Experiment XXXII., *October* 10, 1845.—Malleable iron beam, of the same form as the last, 10 feet 7 inches long, and 10 feet between the supports.

Thickness a = 1·000 inches × 2·75 inches.
" b = ·380 " × 8 "
" c = ·420 " × 4·30 "
Weight of beam = 276 lbs.

Weight in lbs.	Deflection in inches.	Deflection, load removed.	Remarks.
885	·020		
2,606	·050		
4,364	·090		
6,105	·110		
7,835	·140		
9,559	·165	·03	
11,257	·195	·03	
12,999	·220	·04	
14,728	·250		
16,407	·250		
18,108	·290		
19,839	·370		With 21,553 lbs. the deflection increased in four minutes ·025, in the next four minutes ·10, and in four minutes more it had sunk to ·34.
21,553	·475	—	
22,387	·590		
23,046	—	—	Bent laterally upwards of 2·65 inches, when the experiment was discontinued.
∴ Ult. deflection = ·6			

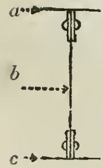
The preceding was the last trial made upon solid beams. They are obviously much inferior in strength to the hollow rectangular girders.

The foregoing experiment was completed on the 14th of October, 1845; and from that time till the beginning of July following little or nothing was done. An abridged report, giving a summary of results from the experiments, was read to the directors of the Chester and Holyhead Railway Company. That report is now before the public; and from the satisfactory nature of the results therein recorded, and those in particular obtained from the tube with the corrugated top, the directors were induced, through the recommendation of Mr. Stephenson, to adopt this description of bridge in preference to others of a less practical character. It was nevertheless considered necessary to make some further experiments on a larger scale, in order to determine the form and proportions of the tubes. For these objects, an entirely new model tube, exactly one-sixth the dimensions of the Britannia Bridge, was constructed; and having arranged the apparatus, the experiments were proceeded with as before.

I have given the foregoing extracts from the experiments instituted to determine the form and strength of the Britannia and Conway Tubular Bridges. As these experiments were the first that had been made upon malleable iron beams, and as they are of the highest importance, considered in relation to the extended application of wrought iron in the construction of buildings, their insertion in this work will not be without use, if they aid in directing the attention of the younger branches of the profession to an effective as well as an economical distribution of a highly valuable material.

Since the above experiments were made, others of equal interest, and bearing more directly upon the subject of beams for supporting floors, have been entered upon. These experiments were made on beams of the annexed form.

Experiment XXXIII.



Dimensions at a = 3½ × ½-inch angle iron.
" " b = ·37 × 16 inches.
" " c = 3½ × ½-inch angle iron.
Weight of beam = 1,380 lbs.
Distance between the supports, 24 ft.; depth, 16 in.

This beam was loaded in the middle progressively with about a ton at a time, until $14\frac{1}{2}$ tons were laid on, when a deflection of 1·6 inches was obtained. In attempting to lay on additional weights a considerable lateral flexure took place, which caused the experiment to be discontinued. All malleable iron beams of this description, whether riveted or rolled, are defective in their resistance to lateral flexure, and require to be not only duly proportioned as regards the sectional areas of the top and bottom flanges, but they also require something to give them lateral stiffness, in order to prevent distortion in that direction, which in many cases takes place before the beam has attained its full powers of resistance.

It will be observed that the beam above referred to had its flanges of similar section and equal areas; whereas the top flange should have been nearly double the area of the bottom one, in order to have equalised the forces of resistance to the transverse strain; had this been the case, and had the beam been fixed, so as to have prevented it from yielding in a lateral direction, it would have borne about 31 tons, for we have by the usual formula—

$$\mathrm{W} = \frac{a\,d\,c}{l} = \frac{7 \times 16 \times 80}{288} = 31 \text{ tons.}$$

Now with half this weight the beam became distorted, partly from weakness on the top side, and partly from want of lateral stiffness. It will not be necessary to record all these experiments, as most of the beams yielded to lateral flexure, owing to the ductility of the material and the facility with which it bends at right angles to the load, unless supported by stays, floors, or brick-arches in that direction. Under ordinary circumstances those supports are not always applicable; therefore it will be safer, and more in accordance with the experiments, to take 60 for the constant in the usual formula instead of 80,* which applies to the tubular form of beams, or to the plate-beams which are secured from lateral flexure. Adopting this number, and assuming that the beams are carefully proportioned in the areas of their respective angle-irons or flanges, the top being nearly double that of the bottom, we have for the above beam, with the top flange doubled in thickness, or a plate riveted along the centre part of the top, the following results :—

---

* Or 75 for the large plate beam, as indicated on page 78.

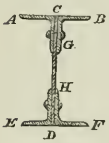
$$\text{W}=\frac{7\times 16\times 60}{288}=23{\cdot}3 \text{ tons.}$$

Which gives the ultimate strength of a wrought-iron beam of the above construction, when subject to lateral flexure.

Experiment XXXIV.—On a beam of wrought iron, composed of a uniform vertical web (7 inches deep, and 7 feet 6 inches long), with two angle-irons riveted to both top and bottom of the rib ; rivets, 4 inches asunder.

Distance between supports, 7 feet.

Fig. 48.



CD = 7 inches.
AB = 4·5 „
EF = 4·5 „
Mean thickness of AB = ·28 „
„ „ EF = ·30 „
„ „ GH = ·25 „

Weight in lbs.	Deflection in inches.	Remarks.
4,216	·10	
8,304	·18	
16,480	·25	
18,667	·36	
22,027	·52	
In five minutes	·54	
24,379	Sunk.	

With the weight 24,379 lbs. the top rib of the beam became twisted.

Experiment XXXV.—On the same beam, rendered straight and uniform.

Weight in lbs.	Deflection in inches.	Remarks.
16,115	·29	The beam was heated by the smiths, and when reduced to its original form it was allowed to cool gradually.
18,355	·36	
19,475	·42	
20,595	·51	
21,715	Sunk.	

With 21,715 lbs. it became bent towards the wall, in a direction in which it was slightly drawn by the lever. Ribs not twisted as before.

The area of the bottom of this beam is 2·8 inches; hence by formula—

$$\frac{2·8 \times 7 \times 60}{84} = 14 \text{ tons} = 31{,}360 \text{ lbs.}$$

That the beam broke with 24,000 lbs. is owing to want of more material in the top to balance the strength of the lower flange.

Collecting together the results of all the experiments on rectangular tubes, they may be tabulated as below, to facilitate comparison.

Summary of Results of Experiments on Rectangular Tubes.

No. of Experiment.	Distance between supports.		Depth of tube.	Width of tube.	Thickness of plate in inches.		Ultimate deflection.	Breaking weight.	Remarks.
					Top.	Bottom.			
	ft.	in.	in.	in.	in.	in.	in.	lbs.	
14	17	6	9·6	9·6	·075	·272	1·10	3,783	Broke by compression.
14A	17	6	9·6	9·6	·272	·075	1·13	8,273	(Reversed). Extension.
15	17	6	9·6	9·6	·075	·142	0·94	3,788	Compression.
15A	17	6	9·6	9·6	·142	·075	1·88	7,148	(Reversed). Extension.
16	17	6	18·25	9·25	·059	·149	0·93	6,812	Compression.
16A	17	6	18·25	9·25	·149	·059	1·73	12,188	(Reversed). Compression.
17	24	0	15·00	2·25	·260	·260	2·66	17,600	Compression.
18	18	0	13·25	7·50	·142	·142	1·71	13,680	Compression.
23	18	6	13·00	8·00	·066	·066	1·19	8,812	Compression. Fin on top.
25	11	0	8·00	1·00	·282	·116	0·75	11,254	Compression.
29	19	0	15·40	7·75	·230	·180	1·59	22,467	Sides distorted. Corrugated top.

These results may be compared together by means of the formula,

$$\mathrm{W} = \frac{\mathrm{A}\, d\, \mathrm{C}}{l},$$

which expresses the ratio of the strength and dimensions for all tubular girders. In this formula, W = breaking load; A = the area of the whole cross-section; C a constant which must be determined by experiment for the particular *form* of the tube; d and l the depth and length as before. Here it will be observed that, finding W by experiment, the value of C determined for different forms of section will enable us to ascertain their comparative strength,—the higher the value of C, the greater the strength for a given amount of material. Hence, deducing the value of C from the different experiments on rectangular tubes, we obtain the following results.

Comparative Strengths of Rectangular Tubes indicated by the Value of C.

No. of Experiments.	Breaking weight in tons or W.	Area section or value of A.	Value of constant C in tons.	Ratio of bottom flange to top flange.	Remarks.
14	1·71	3·20	11·7	1 : 1·01	Fractured by compression.
14A	3·73	5·32	15·3	1 : 3 36	„ extension.
15	1·74	4·04	9·5	1 : 0·53	„ compression.
15A	3·24	4·04	17·8	1 : 1·87	„ extension.
17	8·03	8·00	19 3	1 : 0·60	„ compression.
25	5·05	2·90	28·6	1 : 1·62	„ compression.
25A	3·20	2·90	18·0	1 : 0·61	„ compression.
29	10·13	7·05	21·3		

The results, as exhibited in the above table, show most remarkably the effect of the distribution of the material upon the strength of the girder. Experiment 15, for instance, in which the area of the top flange was only half that of the bottom, gives a constant $=9·5$; but Experiment 15A, with the top flange twice the area of the bottom, gives $C=17·8$, or nearly double the former. With the exception of Experiment 17, which is slightly anomalous, the results are remarkably consistent.

It is also interesting to observe, that the rectangular is considerably stronger than either the circular or elliptical sections. Selecting from each series the experiments upon girders in which the section approximated to the strongest form, we obtain the following ratios for the comparative strengths of the tubes:—

	Mean value of C.
For the cylindrical tubes	13·03
For the elliptical tubes	15·3
For the rectangular tubes	21·5

Further, it will be instructive to compare the ratios of the weight of the tube to the breaking load, as an indication of the comparative strength of the tubes. The following table gives the results reduced from the experiments:—

Comparative Weights and Strengths of Rectangular Tubes.

No. of Experiment.	Distance between supports.	Weight of tube.	Breaking weight.	Ratio of weight to strength.
	ft. in.	lbs.	lbs.	
14	17 6	202	3,738	1 : 18
14A	17 6	384	8,273	1 : 21
15	17 6	255	3,788	1 : 14
15A	17 6	255	7,148	1 : 28
16	17 6	317	6,812	1 : 21
16A	17 6	317	12,188	1 : 38
17	24 0	788	17,600	1 : 22
23	18 6	267	8,812	1 : 33
29	19 0	500	22,469	1 : 50

For further comparison of the results in the last table, it may be observed that the ratio of the weight of a tube to its breaking weight varies directly as the depth of the tube when the length is constant.

Similarly we may compare the results of the experiments upon rectangular beams; the results, reduced as before, give the following comparative strengths:—

Comparative Strengths of Rectangular Beams, indicated by the Value of C.

No. of Experiment.	Breaking weight in tons, or W.	Area of section, or value of A in inches.	Value of constant C in tons.	Ratio of bottom flange to top flange.	Remarks.
30	5·83	6·29	14·3	1 : 1·6	Bent laterally.
31	8·52	7·44	14·5	1 : 1·4	Distorted.
32	10·35	7·59	17·3	1 : 1·5	Bent laterally.
33	14·80	18·9	14·0	1 : 1·0	Bent laterally.
34	10·88	6·3	20·7	1 : 0·9	Top rib twisted.
35	9·69	6·3	18·7	1 : 0·9	Bent laterally.

In consequence of the beams giving way laterally, the above experiments are far from conclusive, and they are of too limited a number to give very definite results. They, however, prove that the top flange should be much larger than the bottom, and, taken in connection with the preceding experiments on similar tubular beams, afford sufficient data for determining their proper form.

ON WROUGHT-IRON TRELLIS GIRDERS.

Since the foregoing experiments were made, I have had occasion to investigate a series of constructions of wrought iron intended for the building erected in Dublin for the Great Exhibition of 1853.*

This building, like its compeer at New York, is erected on nearly the same principles as the Crystal Palace of Hyde Park in 1851. There is, however, this difference, that the beams for supporting the galleries of the Great Exhibition building of 1851 were of cast iron, whereas those of the Dublin Exhibition building are trellis girders, composed entirely of malleable iron of the flat bar, angle L and T forms. Figure 49 (p. 115) is a correct representation of one of these trellis beams.

As some doubts were entertained as to the security of this form of girder, it was deemed expedient, on the part of Mr. Dargan and the Committee, to reassure the public of their safety; and for this object I was requested to visit Dublin and report thereon. To accomplish this object, I had two of the girders supported at the extremities at a distance of about 4 feet asunder; and having covered the centre part of the top with a wooden platform, the girders were progressively loaded with iron in the following manner, until a deflection of 1·62 inches was attained, when the experiment was discontinued.

From the commencement the diagonal stays *a a* exhibited very defective powers of resistance, and, in fact, were of little value in supporting the upper flange exposed to compression. To render these parts effective, they should have been constructed of T or angle-iron, in order to give the required rigidity in their resistance to a crushing force, which was pressing upon them in the direction of the abutments as the deflection of the girders increased. In every construction of this kind, it is desirable that the direction of the forces should be duly considered in order to bring the bearing powers of all the parts simultaneously into action.

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* This great national work was undertaken at the suggestion and sole expense of my friend Mr. William Dargan, the friend of Ireland and the promoter of Irish industry.

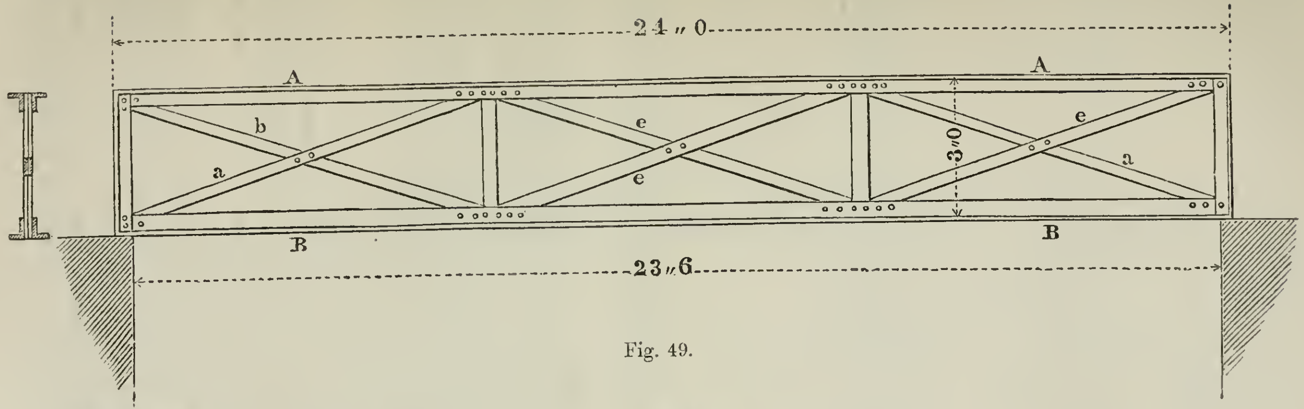
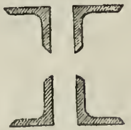
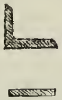


Fig. 49.



Top angle-iron $3 \times 3 \times \frac{3}{8}$.

Bottom ,, $3 \times 3 \times \frac{1}{4}$.



Vertical angle-iron $3 \times 3 \times \frac{3}{8}$.

Diagonal bars $3 \times \frac{1}{2}$.

Weight of girder, 819.

EXPERIMENT TO DETERMINE THE STRENGTH AND SECURITY OF OPEN TRELLIS GIRDERS, AS SHOWN IN THE DIAGRAM (FIG. 49). OCT. 12, 1852.

Weight laid on in tons.	Deflection in inches.	Remarks.
1	—	
2	·107	With 3 tons the diagonals *a a* were slightly buckled, and continued to lose their powers of resistance as the additional weights were laid on.
3	·187	
4	·250	The weights from 3 tons were all placed on the middle part of the girders, or within the two vertical struts *c c*. With 16 tons the diagonal bars *a a* had buckled $2\frac{3}{4}$ inches; with 20 tons, 3 inches; and with 25 tons, $3\frac{1}{4}$ inches.
5	·258	
6	·266	
7	·367	
8	·500	
9	·633	
10	·638	
11	·750	
12	·875	
13	·906	
14	1·000	
15	1·034	
16	1·070	
17	1·088	
18	1·130	
19	1·162	
20	1·185	
21	1·188	These weights were placed nearer the ends of the girders.
22	1·195	
23	1·203	
24	1·218	
25	1·224	
26	1·240	
27	1·280	At this point the diagonal struts or plates *a a* were bent considerably, so as to render them totally inadequate as supports to the upper flange of the girder.
28	1·372	
29	1·440	
30	1·513	26 tons was left on the girders for three days without any apparent increase in the deflection.
31	1·560	
32	1·624	After laying on 32 tons, it was considered advisable to discontinue the experiment, as the girders were considerably crippled in the end diagonal stays *a a*, which were puckered to the extent of 4 inches or upwards.
		Permanent set, ·65 inches.

After a weight of 32 tons was laid on the beam, the ultimate deflection was found to be 1·6 inches; and having removed the weights, the deflection was again taken, when there remained a permanent set of ·65 parts of an inch.

In the construction of girders of this description, there appears to be defects both in respect of the form and of the distribution of the material. Throughout the experiments the diagonal beams *b b* were in a high state of tension, forming, with the bottom flange, the chief element of strength, as a truss supporting the more rigid part of the structure, or top flange A A; and this was accomplished without their receiving adequate support

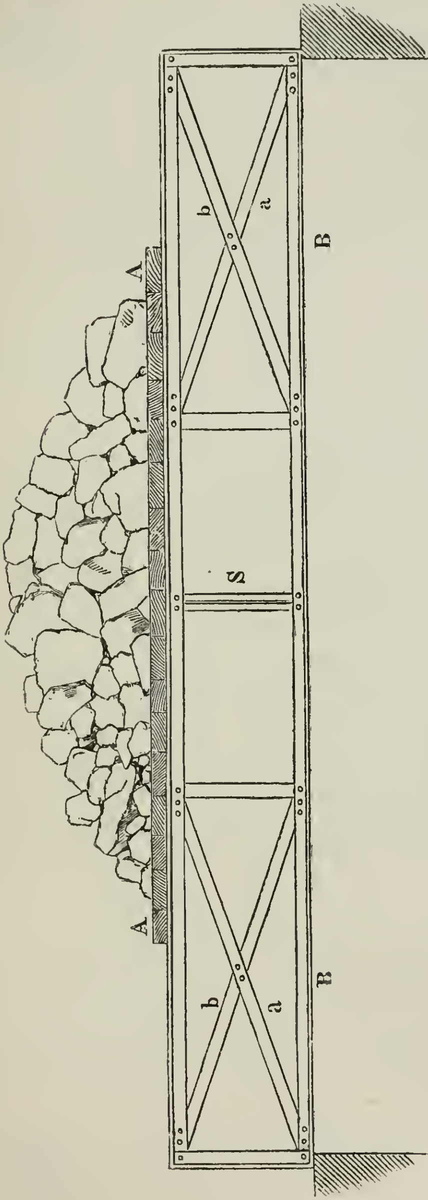


Fig. 50.

from the diagonals a a. On the contrary, the diagonals a a are not only weak and defective from their thin-plate form, but the cross diagonals e e are also inoperative, and become perfectly slack, from the effects of tension upon the diagonal bars b b and the bottom flange, which, in fact, supports the load.

If the diagonals were made of T or angle-iron, so as to act as struts, calculated to resist compression as well as tension, and the two centre diagonals entirely removed, leaving only a vertical T-iron stay in the middle, as at S, fig. 50, we should then have a girder of considerably increased strength, and that without any increase of material. This alteration, with perfect workmanship in the connection and union of the parts, would not only increase the strength, but greatly enhance the value of structures which are in such great demand for buildings of large dimensions, such as crystal palaces for industrial exhibitions, termini for railways, and other buildings.

The diagram, fig. 50, page 117, shows the position of the load, 32 tons, laid upon the girders, and also the suggestion for dispensing with the middle diagonals e e, shown in the original girder, fig. 49. Let us now proceed to determine a formula for calculating the strength of these trellis beams.

We may regard a trellis beam as an imperfect double-flanged beam, where the material of the section is collected at the top and bottom parts. We say imperfect double-flanged beam, because the connection between the top and bottom parts is not so completely maintained as it would be by an unbroken plate or rib. However, with some allowance for this imperfection, we may calculate the strength of these beams on the same principle as the ordinary double-flanged beams. Thus we have

$$\mathrm{W} = \frac{c\,a\,d}{l}, \text{ and } c = \frac{\mathrm{W}\,l}{a\,d};$$

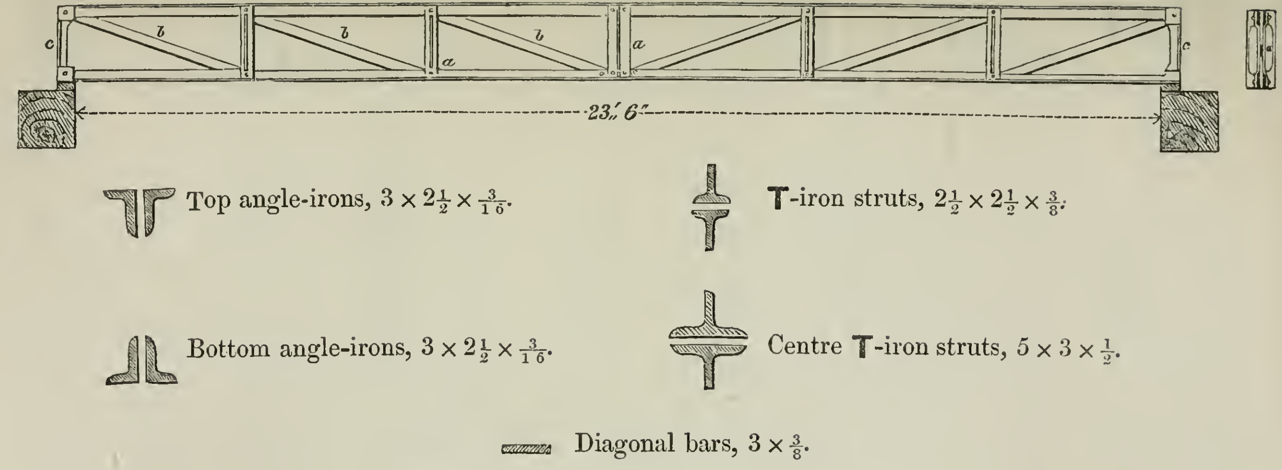
where a is put for the area of the bottom part.

In order to determine the value of the constant for trellis beams, we have, from the Table of Experiments,

$\mathrm{W} = \frac{1}{2}\,(32) = 16$ tons, $l = 23\frac{1}{2} \times 12$, $a = 2{\cdot}8$, and $d = 3 \times 12$; hence we have

$$c = \frac{16 \times 23\frac{1}{2} \times 12}{2{\cdot}8 \times 3 \times 12} = 44;$$

$$\therefore\ \mathrm{W} = \frac{44\,a\,d}{l} \quad \ldots (1).$$



Top angle-irons, $3 \times 2\frac{1}{2} \times \frac{3}{16}$.

T-iron struts, $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{8}$.

Bottom angle-irons, $3 \times 2\frac{1}{2} \times \frac{3}{16}$.

Centre **T**-iron struts, $5 \times 3 \times \frac{1}{2}$.

Diagonal bars, $3 \times \frac{3}{8}$.

Fig. 51.

Weight of girder = 10 cwt. 3 qrs. = 1,204 lbs.
Area of bottom flange = 5·4 square inches.

Hence the value of the constant is only about one-half that for tubular beams.*

Another remarkable instance of the weakness and insufficiency of the trellis girder is that derived from experiments on two of the girders supporting one of the arches of the Central Hall of the Art-Treasures Exhibition at Manchester. As in the case of the Dublin Exhibition, doubts were entertained of their security, and on this occasion, as in the former, I was called upon to give an opinion as to their efficiency for the work they had to perform.

This girder, fig. 51, page 119, it will be observed, is composed of vertical T-iron struts, *a a a*, formed of two T-irons placed back to back, with suspenders *b b b* between them. These are attached by single rivets to angle-irons forming the top and bottom flanges, and the ends of the girder are supported by two cast-iron standards *c c*.

Experiment to determine the Strength of Open Trellis Girders, similar to that shown in Fig. 51. Jan. 1857.

No. of Experiment.	Weight laid on in lbs.	Deflection in inches.	Remarks.
1	2,660	...	
2	5,320	+	
3	8,204	·16	
4	10,864	·29	
5	14,224	·43	
6	16,884	·58	
7	19,544	·74	
8	21,952	·91	
9	24,612	1·23	
10	27,272	1·48	One of the diagonal braces gave way—the rivet tearing through the hole.
11	29,456	1·67	Broke with 13·15 tons on the two girders.

The two girders were placed side by side, at some distance apart, as in the last experiment, and blocks of wood being placed so as to support the ends, they were firmly bolted together, the weights being placed in the centre.

Applying the formula

$$c = \frac{w\,l}{a\,d}$$

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* By some inadvertence, an error was made in the above calculations in the first edition of this work.

in order to determine the value of the constant for this beam, we have

$$w = 6{\cdot}57 + {\cdot}26 \text{ tons}; \quad l = 23\tfrac{1}{2} \times 12.$$
$$a = 4{\cdot}8; \qquad d = 20 \text{ inches}.$$
$$c = \frac{6{\cdot}83 \times 23\frac{1}{2} \times 12}{5{\cdot}4 \times 20} = 17{\cdot}8.$$

This girder, it will be observed, gives a very low constant, and is in every respect a weak and imperfect construction. Its great defect however is the weakness of the suspension bars, at the part where they are attached to the angle-irons, top and bottom, by single rivets. In some of the rods the rivet-holes were so near the edge as to cause the strain to cut out or tear away the iron at that part, so that the girder gave way long before all the resisting powers of the bottom had been brought into operation. Had the girders been duly proportioned and carefully made, they would probably have given a constant at least as high as those for the Dublin Exhibition. All this class of open girders are however weak, uncertain as regards workmanship, and not to be compared with those which have a solid web connecting the top and bottom flanges. We shall however examine the subject more at large, when we come to treat of Lattice Bridges, in the section which has been added to this edition of the work.

For the present we will only add another example of a lattice girder, probably much superior in the distribution of the material to both those we have already given. Fig. 52 is a half-elevation and cross section of the girder which is employed in a bridge on the Ulverstone and Lancaster Railway. It will be observed that the top and bottom flanges are more closely connected together, and more nearly approximate to the condition of a uniform web like that of the plate beam.

The top flange consists of a plate 12 inches by $\frac{1}{2}$ inch, riveted to two angle-irons $3 \times 3\frac{1}{2} \times \frac{1}{2}$.

The bottom flange is similar, except that the flange is only 9 inches broad. At every 3 feet in length, vertical T-irons *a a a* are placed, each being $4 \times 3 \times \frac{1}{2}$; between the ends of these, cross diagonal bars of plate iron *b b b*, 4 inches wide and $\frac{1}{2}$ inch thick, firmly connect the top and bottom flanges together.

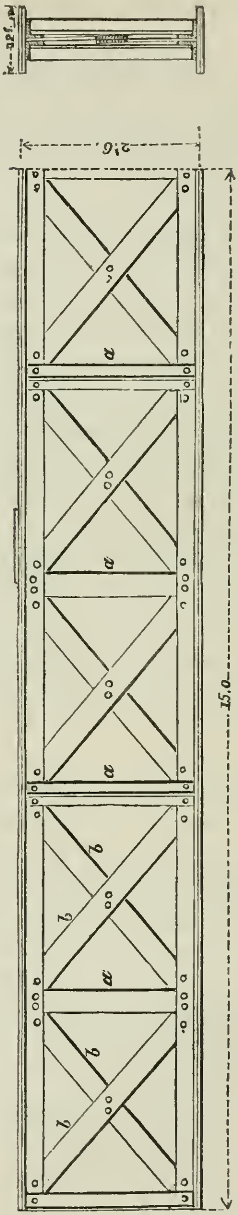

Fig. 52.*

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* This lattice girder was designed by Mr. J. Brunlees, C.E.

The following results were obtained in an experimental trial of the girder :—

Weight on girder in tons.	Deflection in inches.
4	0·00
10	0·15
12	0·20
14	0·25
16	0·25
18	0·30
20	0·30
22	0·35
24	0·40
26	0·40
28	0·50
30	0·50

The girder was not injured with this weight. It will be observed that the deflections are less in this case than in the last, and indicate a stronger girder. Taking the formula for plate beams, we have

$$w=\frac{10{\cdot}5\times 30\times 60}{360}=52{\cdot}5 \text{ tons, the centre breaking load.}$$

And it will be observed that it was tested to within two-fifths of this breaking load; but it is to be regretted that it was not actually broken, in order that the true constant might have been ascertained.

Since these experiments were made, a lattice girder has been carefully constructed for the purpose of ascertaining the proportions of the intersecting stays and tension-bars of a bridge four times the dimensions. The top and bottom flanges were proportioned on the same principle as the plate and tubular girders; this constituted the model of the girder of a railway bridge subsequently erected across the river Tay, near Dunkeld. The experiments become the more interesting as they indicate the proportions, experimentally as well as theoretically, of every diagonal stay and tension bar in that structure. In so far as regards the ultimate strength, there appears to be a great difference between the lattice, the plate, and the tubular girders, with the same area of flanges, top and bottom, and about the same weight of material. Taking, therefore, the proportion as given at page 78, the tubular girder representing 100, and we have by formula $W = \frac{a\,d\,c}{l}$ the following proportions of strength, taking

The tubular or box girder at	100
The plate girder will be	93
And the lattice girder	84

The safe constants, or by calculation the value of c, would then be, for

The tubular or box girder	80·0
The plate girder	74·4
The lattice girder	67·3

Or, for practical purposes, the following constants may be used in the above formula, namely, 80, 74, and 67, which is nearly the value of the three girders respectively.

The reason why so great a difference exists between the box, plate, and lattice girder, is that the solid plate of the two first adds considerably to the resisting powers of the girder both above and below the neutral axis, the sides or middle web of the two first beams contributing largely to their powers of resistance to strain. This is not, however, the case with the lattice or open-sided girder, as nearly the whole of the forces of tension and compression are confined to the top and bottom flanges, the lattice bars imperfectly representing the solid plates; and these facts are the more apparent on comparing the following experiment with those previously made on the tubular and plate girders.

The girder, constructed in the proportion of one-fourth the full size of those in the Tay bridge, was composed of the best ordinary iron. It was 56 feet long, 4 feet deep, and supported on brick piers 52 feet asunder. The top flange consisted of a plate, *a*, fig. 53, $9 \times \frac{3}{8}$, with two side plates, *b*, *b*, $4 \times \frac{1}{4}$ riveted by four angle-irons, *c*, *c*, *d*, *d*, $1 \times 1 \times \frac{3}{16}$ and $1 \times 1 \times \frac{1}{8}$ respectively.

The bottom flange was formed of a plate *e*, $9 \times \frac{1}{4}$, riveted to two side plates, *f*, *f*, $4 \times \frac{1}{4}$, by four angle-irons *g*, *g*, *h*, *h*; and

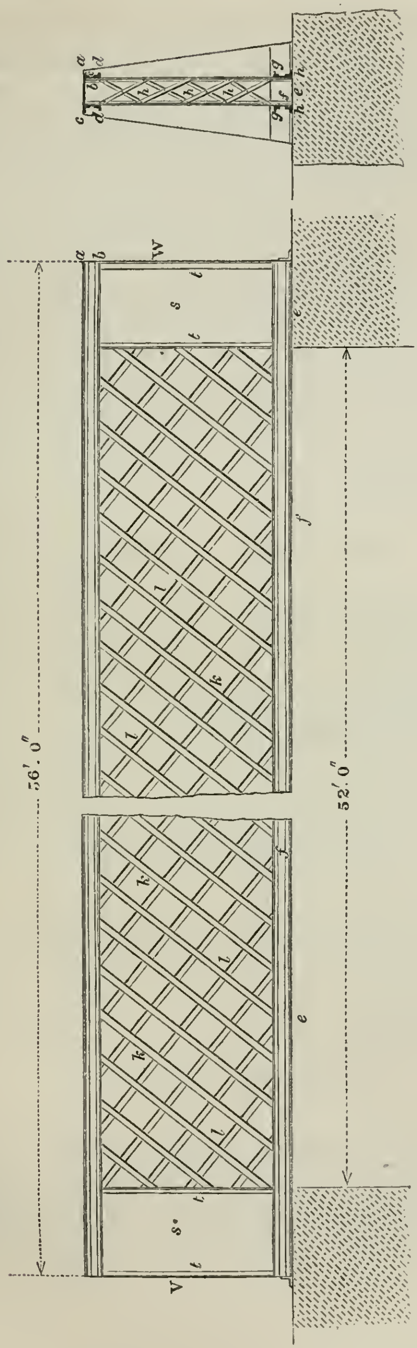

Fig. 53.

uniting the top and bottom flanges were cross diagonal bars *l*, *l*, *l*, *l*, *k*, *k*, *k*, *k*; the former were **T**-irons $1\frac{1}{2} \times 1\frac{1}{2} \times \frac{3}{16}$, the latter and outer ones were plain bars $1\frac{1}{2} \times \frac{1}{4}$. The diagonal **T**-iron stays were riveted together transversely by eight cross pieces to each diagonal *p*, *p*, *p*, &c., $1\frac{1}{8} \times \frac{1}{8}$, to give the girder greater rigidity. At each end of the girder, and on both sides, were plates *s*, *s*, *s*, *s*, 3 feet 7 inches × 1 foot $9\frac{1}{2}$ inches and $\frac{3}{16}$ inch thick riveted to the side plates by the angle-irons *d*, *d*, *g*, *g*. At the extremities were two taper plates, v and w, $11\frac{1}{4}$ inches at the top, and 2 feet $0\frac{7}{8}$ inches at the bottom, 4 feet 4 inches in depth, and $\frac{1}{4}$ inch thick. Riveted to the bottom of these plates were two lengths of 4-inch angle-iron, and to this plate the girder was firmly attached by four lengths of angle-iron on each side, *t*, *t*, *t*, *t*, $1\frac{1}{4} \times 1\frac{1}{4} \times \frac{1}{8}$. The section of the girder was as follows :—

Section of the Top Flange.

	Inches.
1 plate 9 inches × $\frac{3}{8}$ inch	3·375
2 side plates, each 4 inches × $\frac{3}{4}$ inch — extra rivet holes	1·440
2 angle-irons, each $1 \times 1 \times \frac{3}{16}$ . . .	750
2 angle-irons, each $1 \times 1 \times \frac{1}{8}$ . . .	·500
Total section of top . . .	6·065

Section of Bottom Flange.

	Inches.
1 plate 9 inches × $\frac{1}{4}$ inch	2·25
2 side plates, each 4 inches × $\frac{1}{4}$ inch — extra rivet holes	1·44
2 angle-irons, each $1 \times 1 \times \frac{3}{16}$ . . .	·75
2 angle-irons, each $1 \times 1 \times \frac{1}{8}$ . . .	·50
Total section of bottom .	4·94

The beam thus constructed was submitted to experiment; the ends resting on brick piers 52 feet apart. A scale was suspended round the centre of the girder, and the testing proceeded as follows :—

Experiment.—Lattice girder, fig. 53, page 125, submitted to a transverse strain.

Sectional area of the top flange = 6·06 inches.
Sectional area of the bottom flange = 4·94 inches.
Depth of girder 4 feet. Distance between supports 52 feet.

No. of Experiment.	Weight laid on in cwts.	Deflection in inches.	Remarks.
1	12	—	Weight of scale. Deflection not measured.
2	32	·062	
3	52	·125	
4	72	·218	
5	92	·281	
6	112	·406	
7	132	·437	
8	152	·531	
9	172	·593	
10	192	·656	
11	212	·750	
12	232	·875	
13	252	·968	
14	272	1·031	
15	292	1·093	
16	312	1·187	
17	332	1·281	
18	352	1·468	
19	372	1·531	
20	392	1·656	This weight, 412 cwt., was left on 70 hours, and the deflection at the end of this period was found to be 1·875 inches.
21	412	1·718	
22	432	1·937	
23	452	2·000	
24	472	2·125	This weight having been left on for a few minutes, the beam gradually deflected to 2·5 inches, when it suddenly broke across the bottom web near the centre.
25	492	2·250	
26	512	2·406	

From the above it will be observed that the girder broke by tension with a weight of 25·6 tons, and taking the formula $\mathrm{W} = \frac{a\,d\,c}{l}$ as before, we have

$$c = \frac{\mathrm{W}\,l}{a\,d}; \text{ wherefore}$$

$$c = \frac{25{\cdot}6 \times 52}{4{\cdot}94 \times 4} = 67{\cdot}3.$$

From this we derive the constant 67·3 for a lattice girder of this construction, and, comparing this with the constant deduced from the experiments on the tubular girder, we have for the tubular girder 80, and for the lattice girder 67, being in the ratio of 1 : ·84.

Now, if $\frac{1}{8}$-inch plates had been inserted between the top and bottom flanges instead of the diagonal lattice bars, the breaking weight would have been 35·97 tons, and the ratio of that to the lattice as 35·97 : 25·6, or as 1 : ·71. In this case the amount of material in the side plates and in the lattice bars would be as nearly as possible the same.

Immediately following the experiments on the model girder, the bridge which now spans the river Tay was constructed. It consisted of two clear spans of 209 and 141 feet, the girders being of the same proportion as those experimented upon, and 17 feet 6 inches deep. It will not be necessary to give elaborate drawings for this bridge, as the following diagram and section, as shown in figs. 54 and 55, will be sufficient for illustration.

Fig. 54 is an elevation of the bridge, and fig. 55 is a section on an enlarged scale, showing the ends of the main girders with cross beams, 4 feet apart, for carrying a single line of railway.

Having tested the resisting powers of this description of bridge to a transverse strain, it was important to ascertain, by calculation, as minutely as possible, the horizontal strains to which the different parts of the top and bottom were subjected from the centre of the span in the direction of the piers at each end. These were calculated by my late assistant Mr. W. C. Unwin by Mr. J. H. Latham's method as follows:—

THE TAY VIADUCT.

Data of Fixed and Rolling Load assumed for Calculation.

Large span:—

Permanent load = 276 tons = 1·31 ton per ft. run.
Rolling load = 315 tons = 1·50 ,, ,,
Total of two girders 591 tons = 2·81 ,, ,,
Total each girder . 295 tons = 1·40 ,, ,,

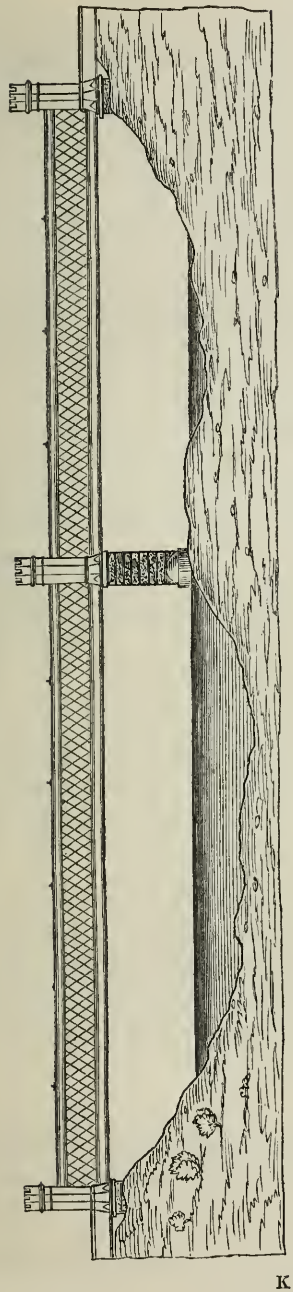

Fig. 54.

Elevation of Tay Viaduct.

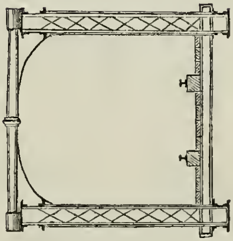

Fig. 55.

Enlarged Section.

Small span:—

Permanent load = 175 tons = 1·25 ton per ft. run.
Rolling load = 210 tons = 1·50 ,, ,,
Total . 385 ,, 2·75 ,, ,,
Total each girder 192 ,, 1·37 ,, ,,

Clear spans 140 and 209 feet.

Depth of girder assumed for calculation at 17 feet.

Calculation of Horizontal Strains on Top and Bottom for a Bridge with discontinuous Girders.

Large span. On one girder:—

Strain at centre, on top compression, on bottom tension, in tons . . . . . = 456 tons.
Strain at 18 feet from centre . . . = 442 ,,
,, 36 . . . . . . = 402 ,,
,, 54 . . . . . . = 335 ,,
,, 72 . . . . . . = 241 ,,
,, 90 . . . . . . = 121 ,,

Small span. On one girder:—

Strain at centre, on top compression, on bottom tension, in tons . . . . = 204 tons.
Strain at 18 feet from centre . . . = 186 ,,
,, 36 ,, ,, . . . = 147 ,,
,, 54 ,, ,, . . . = 82 ,,

In the above, the horizontal strains were computed for separate spans, but there being two spans in this viaduct, the girders on both sides were riveted together over the centre pier and made continuous, which alters the forces as the load passes from one space to the other, as may be seen by the following calculation:—

Calculation of Horizontal Strains on Top and Bottom of one Girder, for a Bridge with continuous Girders, and both Spans fully loaded.

The most economical distribution of material is made when the points of contrary flexure are assumed so as to divide the span into quarters.

Let this be assumed with the lesser span of the Tay Bridge, and the point of contrary flexure be taken at 36 feet from the centre of that span, which nearly fulfils this condition. The strain on each boom at that point is 147 tons. Let, therefore, the girders be made continuous, and a strain of 147 tons tension along the top, and 147 tons compression along the bottom, be introduced over the central pier. For this purpose the joints must be chain-riveted at top, and sufficient vertical rigidity be given to the bottom by cells or otherwise. Subtracting 147 tons from the numbers for discontinuous girders at the several points, we get the corresponding strains for the continuous girders so constructed. Remembering that where the numbers change their sign (+ to −) the strains have changed character, and are tensile at the top, and compressive at the bottom.

To find the point of contrary flexure in the larger span, let x = its distance from the nearer, and y from the farther span; then

$$x\,y = \frac{147\,.\,2\,.\,17\,.\,210}{295} = 3{,}558,$$

which gives $x = 19$ feet very nearly, or the point of contrary flexure 19 feet from the central pier. Hence the effective spans with the girders continuous are—

Larger $= 209 - 19 = 190$ feet.
Smaller $= 141 - 36 = 105$,,

For the centres of these spans we get the following maximum values for the horizontal strains on top and bottom.

Large span :—

$$\frac{266 \times 190}{8 \times 17} = 372 \text{ tons.}$$

Small span :—

$$\frac{144 \times 105}{8 \times 17} = 111 \text{ tons.}$$

Calculations for Strains on the Double Web of Lattice Work on each Girder.

DATA. The angle of the lattices is very nearly 47°. There are two webs to each girder, and the lattices are placed at distances of 4 feet horizontally apart.

In the large span:—		For one lattice web. Tons.
Permanent load on each pin at top	.	$0·437 = w_1$
,, ,, ,, at bottom	.	$0·873 = w_2$
Rolling load on each pin at bottom	.	$1·500 = w'$

(Where in regard to the permanent load one-third is supposed to act at the top of the girder.)

The span is taken in the calculations on the lattice web at 212 feet, to have an exact number of 4 feet spaces.

Hence we get for calculation:

$$w_1 = 0·01208 \text{ of which } \log = \bar{2}·0824222$$
$$w_2 = 0·02415 \quad ,, \quad = \bar{2}·3829550$$
$$w'_2 = 0·04150 \quad ,, \quad = \bar{2}·6180321$$
$$w_2 + w'_2 = 0·06565 \quad ,, \quad = \bar{2}·8172347$$

Maximum compression of Bar No. 26:—

$$= w_1\,(22+14+6-23-15-7) = w_1\,(42-45) = w_1\,(-3)$$
$$+\ w_2\,(18+10+2-27-19-11-3) = w_2(30-60) = w_2(-30)$$
$$+\ w'_2(18+10+2) \qquad = w'_2(30)$$
$$= 0·01208 \times (-\ 3) = -0·03627$$
$$+\,0·02415 \times (-30) = -0·72456$$
$$+\,0·04150 \times (+30) = +1·24493$$
$$\text{Total} \ . \ = \ 0·48410$$

Maximum compression of Bar 27:—

$$= w_1\,(23+15+7-22-14-6) = w_1\,(45-42) = w_1 \times (3)$$
$$w_2\,(19+11+3-26-18-10-2) = w_2\,(33-56) = w_2 \times (-23)$$
$$w'_2\,(19+11+3) \qquad = w'_2\,(33)$$
$$= 0·01208 \times 3 + 0·02415 \times (-23) + 0·04150 \times 33$$
$$= 0·03627 - 0·55550 + 1·40570 = 0·85020$$
$$D_1 = w_1 \times 6 + w_2 \times 7 + w'_2 \times 3 = 0·3661.$$

Maximum compression of Bar 28 :—

$$\left.\begin{aligned}&= w_1\,(24+16+8-21-13-5) = \\ &w_2(20+12+4-25-17-9-1)\\ &w'_2(20+12+4)\end{aligned}\right\}\text{D}_1 = w_1 \times 6 + w_2 \times 7 + w'_2 \times 3, \text{ as before.}$$

No. of Bar.	$w_1\times$				$w_2\times$				$w'_2\times$		Second Differences.
29	25 +	...1 −	20 −	...4	21 +	...5 −	24 −	...0	21 +	...5	w_1
30	26	2	19	3	22	6	23	7	22	6	$-w_2$
31	27	3	18	2	23	7	22	6	23	7	0
32	28	4	17	1	24	8	21	5	24	8	0
33	29	5	16	0	25	1	20	4	25	1	$w_2 + w'_2$
34	30	6	15	7	26	2	19	3	26	2	$-w_1$
35	31	7	14	6	27	3	18	2	27	3	0
36	32	8	13	5	28	4	17	1	28	4	0
37	33	1	12	4	29	5	16	0	29	5	w_1
38	34	2	11	3	30	6	15	7	30	6	$-w_2$
39	35	3	10	2	31	7	14	6	31	7	0
40	36	4	9	1	32	8	13	5	32	8	0
41	37	5	8	0	33	1	12	4	33	1	$w_2 + w'_2$
42	38	6	7	—	34	2	11	3	34	2	$-w_1$
43	39	7	6	—	35	3	10	2	35	3	0
44	40	8	5	—	36	4	9	1	36	4	0
45	41	1	4	—	37	5	8	0	37	5	w_1
46	42	2	3	—	38	6	7	—	38	6	$-w_2$
47	43	3	2	—	39	7	6	—	39	7	0
48	44	4	1	—	40	8	5	—	40	8	0
49	45	5	0	—	41	1	4	—	41	1	$w_2 + w'_2$
50	46	6	—	—	42	2	3	—	42	2	$-w_2$
51	47	7	—	—	43	3	2	—	43	3	0
52	48	8	—	—	44	4	1	—	44	4	0
53	49	1	—	—	45	5	0	—	45	5	w_1

On the next page will be found a table calculated from these numbers, and giving the strain on the bars of one lattice web in the positions indicated by the numbers.

For the strains on the bars in tension we have—

Maximum tension on Bar 30 :—

$$\left.\begin{aligned}&= w_1(19+11+3-26-18-10-2)\\ &+ w_2\,(23+15+7-22-14-6)\\ &+ w'_2(23+15+7)\end{aligned}\right\}\begin{aligned}&= w_1(-23)\\ &+ w_2(3)\\ &+ w'_2(45)\end{aligned}$$

Maximum tension on Bar 29 :—

$$\left.\begin{aligned}&= w_1\,(20+12+4-25-17-9-1)\\ &+ w_2\,(24+16+8-21-13-5)\\ &+ w'_2(24+16+8)\end{aligned}\right\}\begin{aligned}&= -16\ w_1\\ &+ 9\ w_2\\ &+ 48\ w_3\end{aligned}$$

Maximum tension on Bar 28 :—

$$\left.\begin{array}{l} = w_1\,(21+\dots5-26-\dots0) \\ w_2\,(25+\dots1-20-\dots4) \\ w'_2(25+\dots1) \end{array}\right\} \begin{array}{l} = -\,9\,w_1 \\ +\,16\,w_2 \\ +\,52\,w'_2 \end{array}$$

TABLE OF STRAINS ON LATTICE BARS UNDER COMPRESSION.

No. of Bar.	D_2	D_1	Strain in tons on bar.	Remarks.
25	—	0·3661	0·1180	
26	0	0·3661	0·4841	
27	0	0·3661	0·8502	
28	0·0121	0·3782	1·2163	
29	−0·0242	0·3540	1·5945	
30	0	0·3540	1·9485	
31	0	0·3540	2·3025	
32	0·0657	0·3540	2·6565	
33	−0·0121	0·4197	3·0762	
34	0	0·4076	3·4838	
35	0	0·4076	3·8914	
36	0·0121	0·4076	4·2990	
37	−0·0242	0·4197	4·7187	
38	0	0·3955	5·1142	
39	0	0·3955	5·5097	
40	0·0657	0·3955	5·9052	
41	−0·0121	0·4612	6·3664	
42	0	0·4491	6·8115	
43	0	0·4491	7·2646	
44	0·0121	0·4491	7·7137	
45	−0·0242	0·4612	8·1749	
46	0	0·4370	8·6119	
47	0	0·4370	9·0489	
48	0·0657	0·4370	9·4859	
49	−0·0121	0·5027	9·9886	
50	0	0·4906	10·4792	
51	0	0·4906	10·9698	
52	0·0121	0·4906	11·4604	
53	−0·0242	0·5027	11·9631	Sum of strains on last four struts = 50·7234 tons.
54	0	0·4785	12·4416	
55	0	0·4785	12·9201	
56		0·4785	13·3986	

Maximum tension on Bar 27 :—

$$\left.\begin{array}{l} = w_1\,(22+\dots+6-25-\dots-7) \\ w_2\,(26+\dots+2-19-\dots-3) \\ w'_2(26+\dots+2) \end{array}\right\} \begin{array}{l} = w_1\,(-3) \\ +\,w_2\,(23) \\ +\,w'_2(56) \end{array}$$

Maximum tension on Bar 26 :—

$$\left.\begin{array}{l} = w_1\,(23+\dots+7-24-\dots-6) \\ w_2\,(27+\dots+3-18-\dots-2) \\ w'_2(27+\dots+3) \end{array}\right\} \begin{array}{l} = w_1\,(3) \\ +\,w_2\,(30) \\ +\,w'_2(60) \end{array}$$

It will now be seen that this series corresponds with that of the compressions on the struts. Thus—

Tension on Bar 27 = compression on $26 + 53w_2 + 26w'_2$
„ Bar 26 = „ „ $27 + 53w_2 + 27w'_2$

We may therefore calculate the tensions from the compressions, which has been done in the table on the following page.

Check on above calculation.

The strains were taken as $0·437 + 0·873 + 1·50 = 2·81$ tons per foot run on each lattice web.

The strains on one-half a lattice space at each end are supported on the pillars and piers. Hence $52 \times 2·81 = 146·12$ tons equal load on one lattice web.

$\therefore \frac{146·12}{2} \times \frac{1}{\cos 47°} = 107·11$ tons, the strain on 4 struts and 4 ties at the end of the girder.

But strains given in preceding tables $= 50·7234 + 56·4010 = 107·1244$, which, being the same, verifies the long series of calculations above.

Lattice Work of Small Span.

Without repeating in detail the long process which has been gone through for the large span, we may calculate approximately the maximum strains on the lattice bars at the pillars at the extremities of the span.

	Tons.
Total load on each girder . .	= 192
„ on each web . . .	= 96
„ on one pillar . . .	= 48

$$48 \times \frac{1}{\cos 47°} = 70·38 \text{ tons} = \text{total strain in direction of lattices};$$

or, since there are 4 struts and 4 ties, an average strain of 8·8 tons on each of them, that on the ties being somewhat greater, and that on the struts slightly less than this, when the chief part of the load is at the bottom of the girder, as in this case.

TABLE OF STRAINS ON LATTICE BARS UNDER TENSION.

No. of Bar.	D'_2	D'_1	Strain on bars in tons.	Remarks.
34			·2430	
33			·5974	
32			·9518	
31			1·3062	
30			1·6606	
29			2·0150	
28			2·4351	
27	0·0415	2·3590	2·8431	
26		2·4005	3·2507	
25		2·4420	3·6583	
24		2·4835	4·0780	
23		2·5250	4·4735	
22		2·5665	4·8690	
21		2·6080	5·2645	
20		2·6495	5·7257	
19		2·6910	6·1748	
18		2·7325	6·6239	
17		2·7740	7·0730	
16		2·8155	7·5342	
15		2·8570	7·9712	
14		2·8985	8·4082	
13		2·9400	8·8452	
12		2·9815	9·3479	
11		3·0230	9·8385	
10		3·0645	10·3291	
9		3·1060	10·8197	
8		3·1475	11·3224	
7		3·1890	11·8009	
6		3·2305	12·2794	
5		3·2720	12·7579	
4		3·3135	13·3021	Sum of strains on last four bars = 56·4010 tons.
3		3·3550	13·8342	
2		3·3965	14·3663	
1	0·0415	3·4380	14·8984	

PART III.

ON THE CONSTRUCTION OF FIRE-PROOF WAREHOUSES.

The following report was drawn up, in 1844, at the request of Samuel Holme, Esq., of Liverpool, in order to confirm the opinion of that gentleman as to the expediency of erecting all the new warehouses in Liverpool with fire-proof materials. Believing the views contained in this report to be useful and instructive, it is here reprinted for the information of the general reader:—

The serious nature of the late fires at Liverpool, Manchester, and other large towns, has induced an inquiry into the causes of these disasters, with a view to avert their progress, and to adopt measures for the better security of property, and the prevention of a calamity so injurious to public as well as individual interest. In no other description of building have the effects of fire been so severely felt, nor have the provisions necessary for its suppression been so disregarded, as in warehouses used for the stowage of commercial produce in maritime towns.

In the manufacturing districts the same apathy has not prevailed, for in most places fire-proof buildings have been introduced; and considering their complete success, it is surprising that the same system has not been adopted in the construction of warehouses and other buildings appropriated for the reception of merchandise.

When we consider the extent and immense value of property contained in these edifices, it can scarcely be conceived that such a state of things should exist; and, more particularly, amongst a body of men the most active and intelligent in Europe. Such, however, is the case, and we have only to enumerate a few examples to show that a disregard of consequences, or a culpable ignorance of existing improvements, has pervaded the mercantile community for a number of years. This should not be, as the best description of buildings in which the manufactures of cotton, flax, silk, and wool are carried on, are, with few exceptions, almost entirely fire-proof; and upwards of thirty years have elapsed since iron beams, iron columns, and brick arches, were first introduced in the construction of factories, as a security against fire. These facts ought not to have escaped the observation of the British merchant; and yet, in the face of so many examples, with one single exception,* it is only within the last few months that a non-combustible material has been used in the construction of the immense magazines of Liverpool. In other parts of the empire the same laxity of application exists; but the effort so happily made at the port of Liverpool will, it is hoped, extend itself to the metropolis and all the large sea-ports in the kingdom. For these objects, and for the guidance of those who may feel disposed to adopt measures for saving a large rate of insurance, and for the further protection of their property, I would respectfully submit the following observations for consideration:—

In the ages of antiquity we have only a few examples of fire-proof structures; and provided we except the monuments of the early Egyptians, and some of the public edifices of the Greeks and Romans, there are but few instances of buildings so erected as to afford any security against the ravages of fire. During the middle ages some of the Gothic churches and cathedrals were constructed almost entirely of stone;† and, with these exceptions, there appears no evidence of an existing knowledge as to the benefits arising from the use of an entirely fire-proof structure. Probably, the want of cast iron, and the consequent ignorance of its use, was an insurmountable barrier to the development of the fire-proof system; but in the present age these difficulties do not exist; and to neglect the means thus so liberally supplied for the protection of life and property, would augur a want of discernment incompatible with the spirit and enterprise of the age. Latterly, the extension of commerce, and the great value of property which is daily consigned to the keeping of individuals and companies, have produced a different feeling; and, viewing the present engagements of merchants, with the amount of transfer from one hand to another, it is no longer matter of surprise that measures calculated for the better security of property should be imperatively called for, and that in every instance where it is exposed to risk.

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* Messrs. Jevons constructed a fire-proof warehouse at the New Quay, Manchester, ten years ago.

† The cathedral of Milan is constructed entirely of marble and glass.

The general character of warehouses has for ages been the same, the roofs and floors invariably being constructed of timber, with strong girders and wooden props; and these have, in most cases, been so injudiciously placed, as to cause considerable injury to the structure on every occasion when great weights have had to be supported. On referring to the greater number of these erections, it will be found that the props which support the floors have their ends placed immediately under the main beams; and these being successively supported upon each other, with the main beams intervening, the result is, that the fibres are thus completely crushed, particularly in the lower floors, by the superincumbent weight, and in many cases the beams are almost splintered, from the immense pressure to which they are subjected. Even in this imperfect construction the necessary precaution of wooden caps has not in all cases been adopted; and until the introduction of iron columns, with heads and bases covering a large surface of the beam, the timbers were in many instances seriously injured. The use of iron columns, although an improvement upon the old system of building, is nevertheless no security against fire; and it is obvious that no guarantee can be given so long as the structure is chiefly composed of timber, and the openings imperfectly closed by wooden doors and shutters.

From this it is evident, that in order to give perfect security, warehouses should be constructed upon different principles, and these may be enumerated as follows, viz.:—

1. The whole of the building to be composed of non-combustible materials, such as iron, stone, or brick.

2. In order to prevent fire, whether arising from accident or spontaneous combustion, every opening or crevice communicating with the external atmosphere to be closed.

3. An isolated stone or iron staircase (well protected on every

side by brick or stone walls) to be attached to every story; and the staircase to be furnished with a line of waterpipes, communicating with the mains in the streets, and ascending to the top of the building.

4. In a range of stores, the different warehouses to be divided by strong partition-walls, in no case less than 18 inches thick, and no more openings to be made than are absolutely necessary for the admission of goods and light.

5. That the iron columns, beams, and brick arches be of strength sufficient not only to support a continuous dead pressure, but to resist the force of impact to which they are subject by the falling of heavy goods upon the floors.

Lastly, that in order to prevent accident from intense heat melting the columns, in the event of fire in any of the rooms, a current of cold air be introduced into the hollow of the columns from an arched tunnel under the floors.

Adopting the foregoing divisions of the subject, it will be requisite to consider them separately. First. The whole of the building should be composed of non-combustible material, such as iron, stone, or brick.

In the choice of material, much will depend upon locality, and the cheapness at which it can be obtained. In this country the best fire-proof buildings are generally composed of brick or stone, with iron beams and columns, properly framed and held together by rods built into the walls, and brick arches for the floors: which arches are supported by, and spring from, the lower flanges of each beam, and are thus extended in succession on each floor from one end of the building to the other. These arches may be formed either in a longitudinal direction in the line of the building, or transversely, as circumstances may admit. The floors are generally laid with stone-flags or tiles upon the arches, after they are properly levelled, and filled up at the haunches with a concrete of lime, sand, and ashes. The flags or tiles, being well and solidly bedded in mortar, form a durable and excellent floor. In buildings for particular objects, it is sometimes necessary to have wooden floors; and where found necessary, the boards are generally nailed, in the usual way, to sleepers imbedded in the lime-concrete, as before described, or, what is probably better, a pavement of wooden blocks is employed. This description of building, when properly constructed and surmounted by an iron roof, is perfectly impervious to the action of fire; and provided due regard be paid to the selection of a careful superintendent, both owners and occupants may rest satisfied as to the safety of the property.

Secondly. In order to prevent fire, whether arising from accident or spontaneous combustion, every opening or crevice communicating with the external atmosphere should be closed.

These are points which should never be neglected in fireproof buildings. In warehouses, in particular, they are of vital importance; because in rooms or floors where combustible material is stored, nothing tends so much to the security of the building and its contents as a power to shut out and prevent the admission of air. For this purpose, an iron or stone staircase, surrounded by brick or stone walls, and communicating with the different floors by iron doors, should always be attached. This staircase should be easy of approach from without, with a covered opening at the top, and windows at each landing, in order to effect free ventilation, and a ready communication with every part of the building. Warehouses constructed upon this principle will effect almost perfect security; and, in the event of fire, will enable persons not only to approach the locality, but, in case of the casual admission of atmospheric air, the room might be shut up, and the flames smothered, till an effectual remedy was at hand. For these objects, I would strongly recommend the iron doors, frames, and shutters, as constructed and used by Messrs. Samuel and James Holme, of Liverpool, to be fixed in every room. These doors are made of double sheet-iron plates riveted to a skeleton frame, with a stratum of air between, which, acting as a non-conductor, is admirably adapted to the purpose for which they are intended.

Thirdly. An isolated stone or iron staircase, well protected on every side by brick or stone walls, to be attached to every story; and the staircase to be furnished with a line of waterpipes, communicating with the main in the street, and ascending to the top of the building.

Under the second division we have already treated of the staircase and the necessity which exists for having it perfectly distinct from other parts of the building: exclusive of this separation, it will be found still more secure by having a copious supply of water always at command. That supply should not only exist in the street main, but should communicate with every landing by a brass cock and hose, till it terminates in a cistern, with a valve, on the top of the roof. This cistern should be of such capacity as would insure a sufficient supply of water in case of accident to the pipes in the street. The pipes, leather hose, and the requisite discharge cocks, screw-keys, &c., should be kept in good repair, and the hose and screw-keys hung up at every landing, ready for use. These precautions will give additional security to parties bonding goods, as also to the owner of the property in which they are deposited. In addition to the above, it will be advisable that all the cocks, hose and screw-keys be made of one size, and the same as those used by the fire brigade of the town.

Before closing this part of the subject, I would observe, that an exceedingly simple and ingenious apparatus for extinguishing fire has been adopted by Joseph Jones, Esq., of Wallshaw, near Oldham. It consists of a thin copper globe of nine inches diameter, perforated full of small holes, and suspended from the ceiling of the different rooms, either in a mill or a warehouse. Each rose is (in case of need) supplied with water by lines of pipes communicating with the mains in the street. In this form, Mr. Jones is not only in a position to discharge a flood of water into each separate room, but from the peculiar shape of the rose, he is enabled, with a pressure of 200 feet acting upon the apertures, to disperse it to a distance of upwards of 40 feet in every direction. This is a certain and effectual method for extinguishing fire, and might easily be adopted in almost any important structure in large towns, where a supply of water and the necessary pressure can be obtained. Another important feature of this application is the facility and rapidity with which fires can be extinguished. The cocks are all on the outside of the building; and being carefully locked up and marked with numbers corresponding with the different rooms, there is less risk of delay and confusion when an accident occurs.

Fourthly. In a range of stores, the different warehouses should be divided by strong partition-walls, and no more openings to be made than are absolutely necessary for the admission of goods and light.

These precautions become more apparent in every case where large piles of buildings are erected contiguous to each other, and where risk from fire is incurred in the communication of one part of the building with another. The Metropolitan Building Act has provided against accidents of this kind by the insertion of a clause wherein these precautions are insisted upon; and by the introduction of partition-walls which divide the houses, the utmost security is afforded to that description of property. In contiguous buildings these partitions have their full value; and it not unfrequently occurs that the property on each side has been saved from conflagration when a centre building has been completely destroyed: hence the necessity for complete separation in every case where the buildings are contiguous.

In the construction of warehouses these precautions are the more important, from the increased value of the property therein deposited, and the greater risk to which, in some particular cases, they are subject. All warehouses should therefore be carefully separated from each other: and in forming the partition-walls, it might be a great improvement to have an open space of two inches up the middle, with proper binders, for the purpose of ventilation; as air, being a non-conductor, would, in case of fire, prevent the walls from being overheated, and afford a free communication with the atmosphere by the ascending current of air. They should also be built to some height above the roof, in order to prevent the possibility of communication with the adjoining stories, and to effect a complete separation of the different compartments into which they are divided.* To render the different flats or rooms of warehouses secure, it is a desideratum to have as few openings in them as possible. This is the plan adopted in the warehouses of Mr. Brancker, Dublin Street, Liverpool; and they appear to be not only well calculated for the admission and transmission of goods on each side, but having no more windows than are absolutely necessary for the admission of sufficient light to effect the deposition and removal of merchandise, they are exceedingly well adapted for the double purpose of convenience and security. In every situation the iron doors and iron window-shutters already described should be used. It will be observed, that the iron doors and shutters will afford no security unless they be closed and fastened every night before the warehouse is shut up.

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* The Liverpool Building Act has now rendered it compulsory that parapet walls should be built up five feet above the gutters.

Fifthly. That the iron columns, beams, and brick arches should be of strength sufficient not only to support a continuous dead pressure, but to resist the force of impact to which they may be subject by the falling of heavy goods upon the floors.

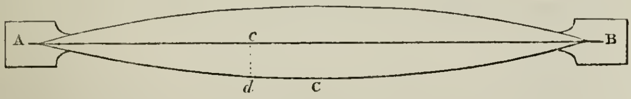
This is one of the most important considerations connected with the security and construction of warehouses; and in order to remove every doubt as to the stability of such a structure, I must refer to my highly talented and respected friend, Mr. Hodgkinson, one of the first authorities in this or any other country on the strength of materials. To that gentleman the public are indebted for a series of theoretical and practical experiments on the strength of beams and pillars, of the utmost value to architects, builders, and engineers. Any person choosing to make himself acquainted with the principles of Mr. Hodgkinson's experiments, and the results deduced therefrom, will find no difficulty in constructing beams and columns of the strongest form, and at the same time ensuring the proportional and requisite strength, accompanied with a great saving in material in all parts of the structure.

On this part of the subject it will be necessary to observe, first, on the structure of beams, that until the publication of Mr. Hodgkinson's experiments, practical men were almost entirely without rule or any satisfactory theory on which to found their calculations on the form and distribution of the material. Now the subject is well understood, not only as regards the strength which is wanted, but also the best and strongest form for resisting the different strains to which they are subjected. In warehouses containing goods these strains are more varied than in factories. In the former, the floors are often loaded, to a great extent, with solid dense material, at other times with light bales; and the lower floors are frequently piled with casks containing mineral substances, which produce not only a great amount of dead pressure upon the beams, but incur the risk of some of the heavier weights falling from some height upon the floor, and thus endangering the security of the structure by the fracture of the beam. These accidents are probably not frequent, but they should be guarded against; and the beams, arches, and columns should not only be calculated to resist the greatest load when operated upon by a dead weight, but the effects of impact produced by a body falling through a given space upon the floor. These calculations should apply to the first two floors of every warehouse, as the heavier descriptions of goods are almost invariably deposited in the lower stories.

Mr. Hodgkinson, in searching experimentally for the strongest section, found that the old practice of making beams with equal ribs—such as recommended by former writers—was exceedingly defective; he proved a proportional between the top and bottom flanges; and the strain being less towards the ends of the flanges, it was reduced to the parabolic form, in order to give equal strengths throughout the whole length of the beams. This was an important discovery; and as warehouse and factory beams are intended to be equally strong in every part, and to sustain the load uniformly distributed, it is necessary to adopt the parabola in the form of the ribs, and to mark their relative properties with the body of the beams and with each other.

In discussing these proportions, Mr. Hodgkinson demonstrates the curvature of the ribs as follows:—Suppose the bottom ribs to be formed of two equal parabolas, fig. 56, the vertex of one of them, A C B, being at C,

Fig. 56.



then, by the nature of the curve, any ordinate, dc, is as $\mathrm{A}c \times \mathrm{B}c$; the strength of the bottom rib, therefore, and consequently that of the beam at that place, will be as this rectangle. It is shown, too, by writers on the strength of materials, that the rectangle $AC \times BC$ is the proportion of strength which a beam ought to have to bear equally the same weight everywhere, or a weight laid uniformly over it. From this it would appear that the forms laid down by Mr. Hodgkinson were rightly devised, and a great saving, of not less than three-tenths, was effected in the quantity of material used.

Having pointed out the strongest form of beams, as applied to fire-proof buildings, it will be necessary in this place to refer to their strength, and to inquire into the nature of the strains to which they are subject. It has already been stated, that iron beams in warehouses have two distinct forces to contend against, that of direct pressure and that of impact; with the former there is no difficulty, but the latter involves a proposition on which mathematicians are not agreed. For practical purposes we may, however, suppose a case, such as a large cask of molasses, or box of heavy mineral substance, equal to one ton = 2,240 lbs., falling from a height of six feet upon the floor. Now, according to the laws of gravity, a body falling from a state of rest acquires an increase of velocity, in a second of time, equal to $32\frac{1}{5}$ feet, and during that period falls through a space of $16\frac{1}{10}$ feet: this accelerated velocity is as the square roots of the distances; and a falling body having acquired a velocity of 8·05 feet in the first foot of its descent, and 6 feet being the height from which a weight of 1 ton is supposed to fall, we have $\sqrt{6} \times 8{\cdot}05 = 2{\cdot}449 \times 8{\cdot}05 = 19{\cdot}714$ for the velocity in a descent of 6 feet. Then, $19{\cdot}714 \times 2{,}240 = 44{,}159$ lbs., or nearly 20 tons, as the momentum with which the body impinges on the floor. In the present state of our knowledge, this momentum may probably be taken as the measure of the force of impact, but it is to be remembered that the resistance of a beam to impact is very different from its resistance to transverse pressure.* Having these forces to resist, it will be necessary to guard against them, and to make the beams, columns, and arches in the lower floors of such strength as will resist the blow, and neutralise its effect upon the floor.

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* Since the above was written, the Commissioners on Railway Structures have made an experimental inquiry into this subject, and as the results bear immediately upon the point in question, it will be instructive to give a brief abstract of them here. The object of the experiments was to ascertain the effect of additional loads spread uniformly over the beams in increasing their power of bearing impacts from the same ball falling through different heights. The beams were of Blaenavon iron, No. 2, cast to be 14 ft. 6 in. long, and 3 in. square.

Mean weight of beam	410·7 lbs.
Mean weight of beam between supports	382·0 lbs. nearly.
Distance between supports	13 feet 6 inches.
Weight of ball	303 lbs.

No. of Experiment.	Additional load on beams in lbs.	Height of fall necessary to break the beam.	Velocity of impact answering to that height.	Momentum or W V.	Remarks.
		Inches.			
1	None	28½	12·358	3,744	The set from the impacts on the loaded beams was very great, but it did not appear to injure their strength more than in ordinary cases.
2	Lead, 4 lbs. weight in centre	33	13·301	4,030	
3	28 lbs. in centre, no lead	42	15·005	4,546	
4	166 lbs. spread over, + 4 lbs. lead in centre	48	16·042	4,860	
5	389¼ lbs. spread over beam, 4 lbs. lead in centre	48	16·042	4,860	
6	389 lbs. spread over, no lead	48	16·042	4,860	
7	391·2 lbs. spread over, 4 lbs. lead in centre	66	18·810	5,699	
8	956¼ lbs. spread over, 4 lbs. lead in centre	60	17·935	5,434	

From the above it will be seen that the power of resisting impact increases with the permanent load upon the beam; the greater the weight at rest upon the beam, the greater must be the momentum of a striking body in order to break it. This is satisfactory, as it diminishes the risk from falling weights in warehouses; the more nearly the weight upon the floors approaches the point at which danger begins, the greater is their power of resisting sudden impacts. Further, the deadening influence of the small piece of lead in experiment 2 is strictly analogous to that of the wooden flooring of buildings.

To compare the above numbers with the power of resistance to transverse strain, we may select from the reports of the Commissioners the following table of results on precisely similar bars.

Transverse Breaking Weight of Bars of Blaenavon Iron, No. 2, 13 *ft.* 6 *in. between supports; results reduced to those on bars exactly* 3 *in. square.*

No. of Experiment.	Weight of bar between supports (unreduced).	Centre breaking weight in lbs.	Ultimate deflection in inches.
1	380·08	2,698	4·863
2	378·9	2,671	4·391
Mean	379·49	2,685	4·627

Although the iron beams and arches of a fire-proof floor may be sufficiently elastic to resist an impinging force, such as above described, it is still advisable to adopt other precautions, such as the bedding of timber along the top of the arches,* or to form the two lower floors entirely of wooden boards (three-inch plank) securely nailed to sleepers embedded in concrete. This plan would give additional security, by the transmission of the impinging force over a larger surface; and, under these circumstances, the concussion would be made in the first instance on a soft elastic substance, before it could act upon the more rigid materials of iron beams and brick arches.

In order, however, to remove all doubts as to security, it will be advisable to have stronger iron beams and columns in the two lower floors: and having computed these strengths, they will probably be found nearly correct in the ratio of 12 to 9. If on this data we take the breaking weight of a beam, as suitable to the upper stories of a warehouse, at 22 tons, those of the lower stories would require to be 29·32, or nearly 30 tons; and the columns, although less liable to fracture, will nevertheless be greatly improved by the introduction of a proportionate thickness of metal.

Having, to the best of our ability, established the fact of perfect security in the use of iron beams and arches, the next point of inquiry will be as to the strength and proportion of the columns. But before treating of this part of the subject, it may be proper to advert to the tie-rods, which are built into the walls and arches, and should unite the walls and girders as a species of net-work. These tie-rods are of great value, as they resist the strain of the arches, which, acting through their

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Comparing the mean results of these two experiments with the power of resisting impact by a similar bar, not loaded, as evidenced by the first experiment in the previous table, we find that the transverse is to the impactive strength as 2,685 : 3,744 or as 1 : 1·39. Similarly when the bar subjected to impact is loaded with 28 lbs. in the centre, the transverse is to the impactive strength as 2,685 : 4,546, or as 1 : 1·69; and when 391 lbs. is spread uniformly over the bar, the transverse is to the impactive strength as 2,685 : 5,699, or as 1 : 2·12.

* Since the above was written, I have been informed that the Act of Parliament for the regulation of fire-proof buildings does not admit of any timber whatever. In such case, I would advise the beams so to be made one-half stronger.

line of tension, not only secure the walls from being thrust out, but also retain the beams in the position best adapted to sustain the load. The usual practice in these districts is to have five lines of $\frac{3}{4}$-square rods in a width of 30 feet; two lines are embedded in the wall, and the remaining three built into the arches: this is considered a perfectly secure building. But it must be borne in mind that cotton mills are not subjected to heavy loads; and instead of five tie-rods of $\frac{3}{4}$-inch square, a warehouse should have seven lines of rods, each $1\frac{1}{4}$ inch square. This will give a sectional area of about 11 inches in 30 feet, which, taken at 25 tons to the square inch, will give a resisting tensile force of 275 tons. In factories, the resisting powers of the tie-rods seldom exceed 100 to 110 tons, which is under 4 tons to the foot, whereas the resisting forces in warehouses should not be less than from 9 to 10 tons to the foot.

In the construction of fire-proof buildings, it is not only necessary to secure the ends of the beams by tension-rods embedded in the walls, but the arch-plates or 'skew-backs' at each end should also be built into the wall; and this plate, as well as the ends of the beams, slightly raised above the level of the column, in order to allow for the settling of the walls, which invariably takes place as the weight increases in their ascent. For the strongest form and best position of columns supporting heavy weights, we must again refer to Mr. Hodgkinson, as the very first authority. In his valuable treatise on the strength of pillars of cast iron and other materials, published in the Philosophical Transactions, Part II., for 1840, and for which he received the gold medal of the Royal Society, will be found some of the most interesting and most useful experiments yet given to the public.

From these researches it will be necessary to make some extracts, in order to ascertain the laws connecting the strength of cast-iron pillars with their dimensions, and to determine the best and strongest form adapted to the support of heavy weights. The first experiments were made upon solid uniform pillars, mostly cylindrical, with their ends rounded, in order that the force might pass through the axis; the next were of the same dimensions, with flat ends at right angles; and others, again, with one end rounded, and the other flat to the axis. They were broken at various lengths, from five feet to one inch (some with discs turned flat), and form a series of most interesting results. The pillars with discs give a small increase of strength above those with flat ends; but the approach to equality between the strength of pillars with discs, and those of the same diameter and half the length, with ends rounded, was nearly alike. The conclusion, as Mr. Hodgkinson observes, is, therefore, 'that a long uniform cast-iron pillar, with its ends firmly fixed (whether by means of discs or otherwise), has the same power to resist breaking as a pillar of the same diameter and half the length, with the ends rounded or turned, so that the force would pass through the axis.'

Mr. Hodgkinson, in the first experiment, gives the strength of cast-iron pillars with both their ends rounded and both flat; subsequently he experimented upon those with one end rounded and the other flat, and in some cases with discs; and their results being placed between those from the pillars with round and flat ends, gave the strength in a constant ratio, as under:—

Pillars.	Breaking weight in lbs.				
Both ends rounded . . .	143	3,017	7,009	7,009	16,493
One end rounded and one flat .	256	6,278	13,499	13,565	33,557
Both ends flat	487	9,007	20,310	22,475	—

These pillars, in each vertical column in this abstract, are of the same length and diameter; the strengths, therefore, in three different cases, reading downwards, are as 1, 2, 3, nearly, the middle term being an arithmetical mean between the other two terms.

Mr. Hodgkinson therefore found, by other experiments upon timber, wrought iron, steel, &c., that those, as well as every other sort and description of material, followed (as regards their strengths) the same laws; and that the strength of a pillar with one end round and the other flat is always an arithmetical mean between the strength of pillars of the same dimensions with both ends rounded and both flat.

These are facts which should on no account be mistaken in the construction of fire-proof buildings; and it will be well to impress forcibly upon the public mind, that the principle is the same, however much they may vary in their ratio of strength.

In treating of the strength of columns, I have endeavoured to establish principles which are not generally known, but which are proved to be fixed and determined laws, affecting the increase or diminution of strength, according as the ends are made round or flat. In order, therefore, to avoid error in the construction of buildings, adapted for the support of heavy weights, it will be of some value to know, that the strength of pillars can be increased, according as their ends are shaped, in the numerical ratio of 1, 2, 3.

Having investigated the subject at some length, it may be necessary, before closing the report, to advert to a circumstance which appears to excite alarm, and increase the fears of individuals, respecting the safety of iron beams and brick arches as a perfectly fire-proof structure. It has been alleged, that in case of fire in any of the lower rooms in a warehouse, the intense heat generated by rapid combustion might melt the iron columns, and bring the whole edifice to the ground.* This is a possible, but a very improbable case, as an event of this kind could never happen, provided the precautions enforced and inculcated in this inquiry be duly and properly observed. It is true, that negligence of construction on the one hand, and want of care in the management on the other, might entail risk and loss to an enormous extent; but it is no argument to say, that a warehouse built like a funnel, and provided with all the elements of conflagration, is attended with risk, when it is well known that a perfectly secure and perfectly sound fire-proof building can be erected, free from all the perils above enumerated. In my own mind, there is not the shadow of a doubt as to the security of such a structure; and I do not hesitate to assert, that a well-built and properly-arranged fire-proof warehouse can not only be constructed, but may be made to entail upon the commercial and manufacturing communities of this country an important and lasting benefit.

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* There is only one instance which has come to my knowledge of a fire-proof building being injured by the melting of the columns, and that was at the works of Messrs. Sharp, Roberts, and Co., Manchester, where the pillars were fixed between the boilers of a steam engine; and having a large quantity of wood piled round them on the top of the boiler, for the purpose of drying, the heat became so intense as to cause them to bend, and ultimately to break. In this case the front of the boiler-house was open, with a thorough draft direct across the building, which generated a most intense heat, and caused the whole room to act as a reverberating furnace. Viewing the subject in this light, it cannot be considered analogous to a warehouse efficiently secured against the admission of atmospheric air.

For some time after the date of the foregoing report (1844), we had little or no knowledge of the superior resisting powers of wrought iron in the shape of beams. Since then, important changes have been effected, and new elements of construction have come into operation. It is true that wrought iron had been used for various purposes, and even beams of that material had been made on a small scale; but, excepting in ships, it was applied to a very limited extent. Its elasticity, ductility, and powers of retention were almost entirely unknown, until the subject was investigated on a large scale, with a view to devise means for carrying the Chester and Holyhead Railway across the Conway and Menai Straits. Our knowledge of the properties of wrought-iron beams and girders may be considered as still very imperfect, and confined within an exceedingly narrow compass. The discovery and development of new principles in the application of wrought-iron beams, as deduced from these experiments, has rendered some of the instructions given in the report almost nugatory: but sufficient still remains to render the observations useful to the professional practitioner and the general reader. Architects and builders may safely refer to it for general information in every description of fire-proof buildings, whether constructed with wrought or cast-iron beams.

It will not be necessary further to enlarge upon the subject, but simply to direct attention to the following details, which may be entitled to consideration in the application of wrought-iron beams for the support of the floors of buildings. In another place I have given the necessary information as regards construction, strength, and other properties of iron, as a material for building; and I must now direct attention to the erection, and to the principle upon which I consider that these adjuncts should be attached and securely united to the structure.

In every building, whether intended for a factory which contains machinery, a magazine, a warehouse for sustaining heavy weights, or for a public edifice, one important consideration presents itself—namely, a direct combination of all the parts, and a perfect union of the floors with the surrounding walls. In fact, in order to ensure strength and security in every part, there should be a balance; or, if I may use the expression, the structure should be in equilibrium as regards its powers of resistance to the respective weights which its parts have to support, or the strains which they have to sustain; or, in other words, the resistance of its various parts should be proportioned to the loads which they have respectively to bear.

By keeping these objects in view, we should not only economise a great deal of useful material, but we should harmonise the parts, and ensure more correct proportion, both as regards strength and symmetrical effect. Nature works in this way; and we may venture to affirm that, in the vegetable as well as in the animal creation, every part is fitted and proportioned according to the office which it has to perform. In nature, taken as a whole or in part, the greatest harmony in form and structure prevails, and the utmost exactitude in arrangement and distribution is everywhere apparent, in order to obtain the greatest possible strength with the least expenditure of material. It is thus that the great Architect of nature works; and we have only to study the conditions and forms of construction in the natural world, to arrive at the true principle for practice in the arts.

The construction of fire-proof buildings is of two kinds: those entirely composed of iron, brick, or stone; and those composed of iron beams and columns, with brick arches. The exterior walls may be formed of either material, as the conveniences of locality or circumstances may admit; but the floors, with which the present inquiry is more directly concerned, will require a separate and a more detailed description.

The floors of buildings, when formed of brick arches, require consideration in regard not only to the weight of the material, but also to the nature of the arches, and the lateral thrust which they exert upon the sides of the beams and the gable-walls upon which they abut. In mills for the manufacture of cotton, silk, flax, and wool, the span of the arches varies from 9 to 10, and sometimes to as much as 11 feet. These arches

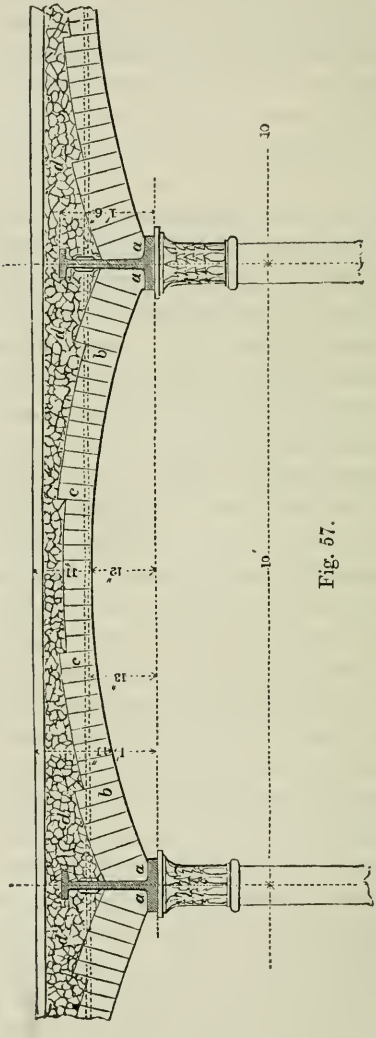

Fig. 57.

are generally composed of the segment of a circle, with a rise or versed sine in the top of about one-twelfth the length of the chord. One-tenth, however, is safer in practice for mills, and one-eighth for warehouses where heavy goods are stored. These proportions will vary, however, according to circumstances, the nature of the strains, and the uses for which the building is intended.

In factories the arches are composed of good hard brick, with the springers moulded for the purpose, as at *a a a a*, fig. 57.

The parts of the arch from the springers to *b* are of common bricks, 9 inches deep; the parts from *b* to *c* are of three-quarter bricks, or bricks 7 inches deep; and the middle or crown of the arch is of bricks on edge, 4½ inches deep. All these bricks are moulded to suit the curve of the arch; but if not so moulded, they are generally wedged with thin slates and mortar between the joints on the top, in order to give them a solid bearing. After the arches are turned and properly secured, the haunches or spans next the beams are filled up and levelled with a concrete of lime and ashes, as shown at *d d d*. This, with a little plaster, gives a smooth surface to the floor, on which are embedded the large flags or tiles, as circumstances may require. The arches of warehouses are generally made stronger; they have a greater depth of brick, and a greater rise in the versed sine of the arch, in order that they may not only support greater weights, but resist the force of impact, and those concussions to which they are subject, from the falling of heavy bales and packages when piled to a considerable height above the floor. The utmost care should, however, be observed in rendering these parts of the structure as light as may be consistent with the required strength, more particularly when cast-iron beams are used, as every additional ton will increase the risk of failure in the event of any unforeseen defect in the castings.*

In the designs for fire-proof buildings, other considerations are requisite besides those relating to the beams and arches, viz., the strength of the columns, base-plates, and stay-rods, for retaining the beams in line, and for preventing them from warping before the arches are turned. In every building of this kind it is important to have the tie-rods as low down as possible. In fact, the proper position would be along the bottom flanges, forming a series of chords or tie-bars to the respective arches. This cannot, however, be done without disfiguring the ceiling, and giving to the structure an appearance of complication, as if it were held together by a series of rods. To avoid these distortions, the tie-bars are embedded in the arches; and being firmly keyed at every beam, and also to the columns, they are effectually concealed from view, and are thus rendered nearly as efficacious as if they were extended along the bottom of the beams. To give the floors additional strength and security, and to attain an effectual combination of the beams with the walls on each side, and also with the gables, a tie or bond rod of the same strength as those in the middle is embedded longitudinally in the middle of the side-walls; and this being securely fixed and keyed to the end of each beam, a complete framework is thus formed, which binds the whole into one mass, and gives greatly increased strength and security to the structure. The following plan of the iron tie-bars, &c., at one end of one of the most recently erected fire-proof cotton mills, will give a correct idea of the connexions and attachments of the beams and columns binding each other, and of their combination with the walls of the building.

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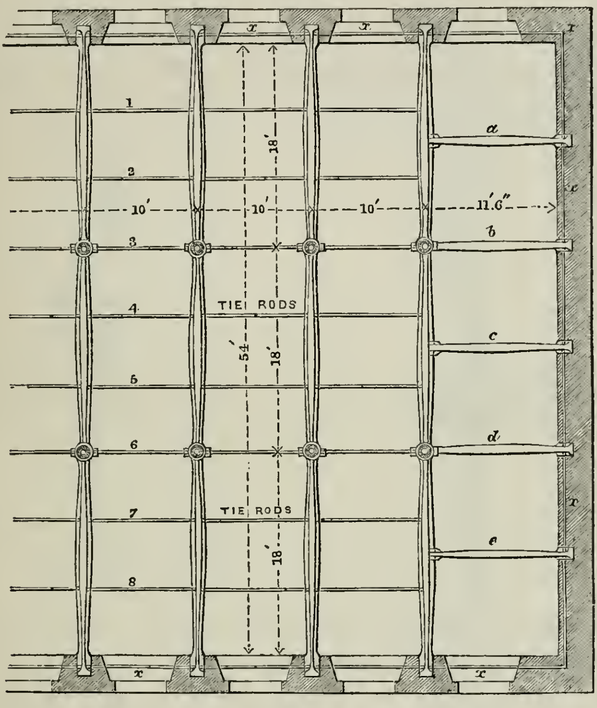
* In one of the most extensive mills in this country, that of Titus Salt, Esq., Bradford, hollow bricks have been used for the floors: these combine great lightness and security in every room of the factory.

In the following figure it will be observed that the tie-rods 1, 2, 3, 4, &c., terminate at the last line of beams and columns, about 10 feet from the gable-end of the mill. The object of their termination at this point is to make room for the short longitudinal beams *a*, *b*, *c*, *d*, &c., which are placed with their ends resting on the gable-wall, and on the flange of the cross-beams, A, B, and C, which are made stronger for that purpose. These beams support arches at right angles to those across the mill; and having their ends resting upon the last cross-beam and the gable-wall, as before described, they form a strong and powerfully resisting abutment to the thrust of the transverse arches running from one end of the building to the other.

The rods *x x x*, are cottered into the ends of each of the beams, and form a bond, when built into the walls, which ties the parts together, and unites the flooring-beams and the side-walls, as well as the gables, into one solid and compact mass.

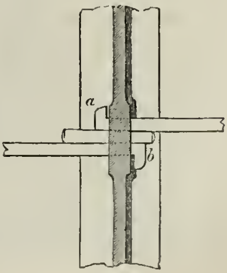
The principle of construction just described applies exclusively to cast-iron beams, which, although somewhat analogous to those constructed of malleable iron, are nevertheless different in form and in the mode of attachment to the beams. In cast iron the rods or bars vary from three-fourths to seven-eighths of an inch square, and are fixed to the beams by a key or cotter, as shown in fig. 58. This connexion is probably the best, as the rods are not weakened by cotter-holes, and are simply formed into single gibs, with shoulders pulling against the beam on each side, and a key between, as represented at *a b*, fig. 59.

Fig. 58.



The heads of the columns are fixed in the same way, the rods running right through an oblong hole, and keyed vertically, or the reverse of those through the sides and ends of the beams. Figs. 60 and 61 are correct representations of this part of the structure; and also show the method usually adopted for supporting the ends of the beams on the collar of the column, the top or socket part of which is removed,

Fig. 59.



for the purpose of showing that portion in section. The ends of the beams in this arrangement, it will be observed, embrace the column, and are held together by wrought-iron hoops, fitted on the projecting horns at *c*, *d*, fig. 60. The iron tie-rods, *e*, *f*, pass through the columns, with an iron key between, in the vertical direction, similar to those shown through the beam at *a*, *b*, fig. 59.

The columns and base-plates will require a short notice respecting their form, and the way in which they are supported from one to the other on the separate floors.

Fig. 60.

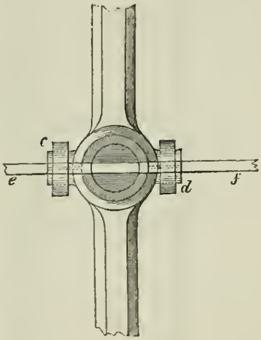
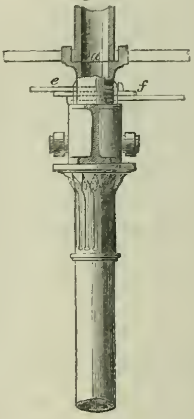


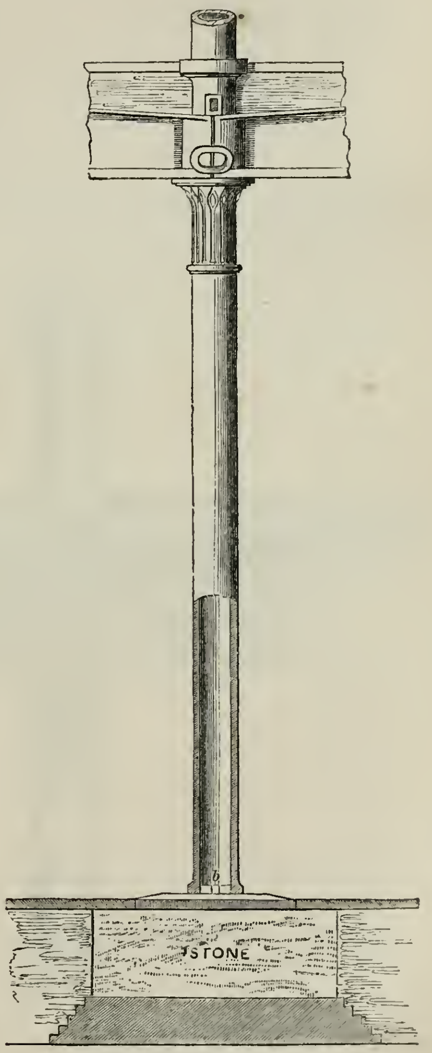
Fig. 61.



The annexed drawing, fig. 62, exhibits the column, in connexion with the foundations and base-plate, as also the tie-rod and the lower end of the superincumbent column *a*, which fits into the socket prepared for its reception in the one below. This is shown in section at *a*, fig. 61; and the base-plate, which varies from 2 feet to 2 feet 6 inches square, according to the height and strength of the building, is likewise shown in section, with the projecting cross of the base-plate, which fits into the hollow of the pillar at *b*, fig. 62. The base of the lower column should in every case be considerably enlarged, and the ends faced in the lathe; the base-plate which receives it should also be faced. This is the more necessary, as it gives an even surface for the purpose of levelling the plate, and maintaining the vertical position of the column. The same operation is performed on the upper end of the socket, and on the bottom of each succeeding column.

In mill-architecture, the columns seldom exceed 8 inches in diameter, with a thickness of metal varying from 1$\frac{3}{8}$ inches at the bottom to $\frac{5}{8}$ths at the top,—the columns, under these circumstances, being of the same diameter from top to bottom. These are the dimensions for columns supporting beams of 24 or 25 feet span. In other cases, where the span of the beams is not more than 18 feet, the pillars are reduced to 6½ or 7 inches in diameter.

Fig. 62.



Having treated this part of the subject more in detail than I at first intended, and having given a number of practical illustrations, deduced from a lengthened experience, I would now direct attention to a construction precisely similar, with the exception, that wrought-iron beams are used instead of cast-iron ones, and thin iron plates instead of brick arches. For this latter construction, the columns, base-plates, &c., are in every respect similar to those used for cast iron, the only difference being the connexion of the ends of the beams with the column and the tie-rods, which, instead of being square, as used in the former case, are flat bars, riveted to the top flange, as shown in the annexed sketch at *b*, *b*, fig. 63, which represents a plan of the beams, and their connexion with the columns, as at *a*, *a*, fig. 64, and *e*, *e*, fig. 65. The flat tie-bars, as shown at *b*, *b*, fig. 63, are fixed by rivets, as already

Fig. 63.

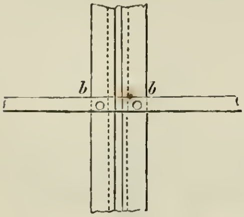


Fig. 64.

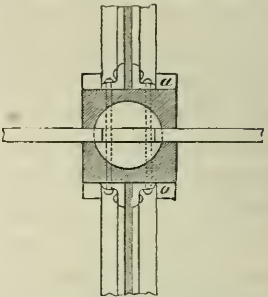
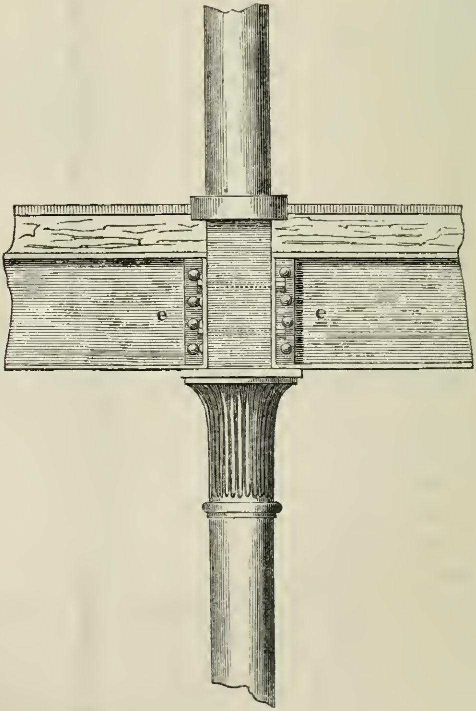


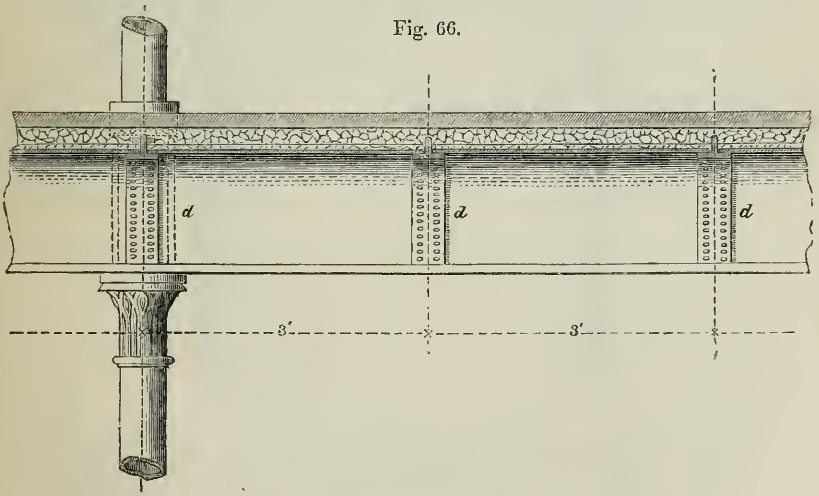
Fig. 65.



noticed, to the top flange, and placed at the same distance apart as those already exhibited in fig. 58. Those in the walls, which form the bond-plates, are riveted to the ends of the beam in the same manner, and thus form the connecting link between the ends of the beams and the piers on which they are supported.

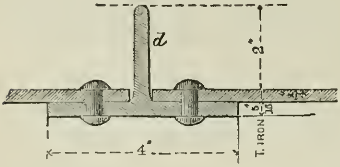
In order still further to elucidate the advantages of this construction, let us suppose a fire-proof building, of any given length, and 48 feet wide, having only one row of columns in the middle, and wrought-iron beams, 22 feet 6 inches long, extending from the centre columns to the walls on each side. Now, on this plan, if the beams were rolled according to the form given in fig. 37, p. 83, they would be equal to a breaking weight of 13 tons in the middle, or 26 tons equally distributed over the surface of the beam. This would almost give sufficient security for the support of brick arches of the usual construction. But assuming wrought-iron plates one-fourth of an inch thick, 3 feet wide, and bent in the segmental form of an arch, 10 feet wide, with ribs of ⊤ iron, as shown at *d*, *d*, *d*, *d*, *d*, figs. 66 and 67, and in section, as represented in figs. 68 and 69 also at fig. 70, where one whole arch is shown, with the filling up of the haunches, *a*, *a*, *a*, *a*, &c., with concrete, to level the

Fig. 66.



surface, preparatory for the reception of the floors;—this will be found a floor of exceedingly light construction, perfectly fire-proof, and containing all the elements of strength and rigidity which can possibly be attained by cast-iron beams and brick arches.

Fig. 67.



The advantages peculiar to this construction are such as to entitle it to more than ordinary consideration; and conceiving that the time is not far distant when wrought-iron beams and plates will form the principal supports of the floors of fire-proof buildings, I have deemed it worthy of the following illustrations, as represented in fig. 71, which exhibits a ground-plan of a part of one of the floors of a cotton or flax mill, and the position of the transverse wrought-iron beams, columns, tie-rods, and arch-plates, as they are respectively shown at *b*, *b*, *b*, and c, c, c, &c. The flat bar tie-rods, which

Fig. 68.

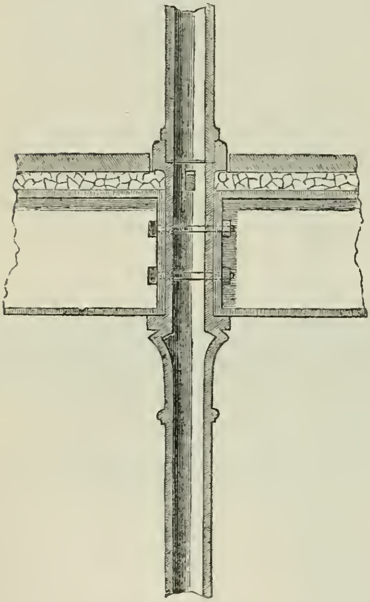
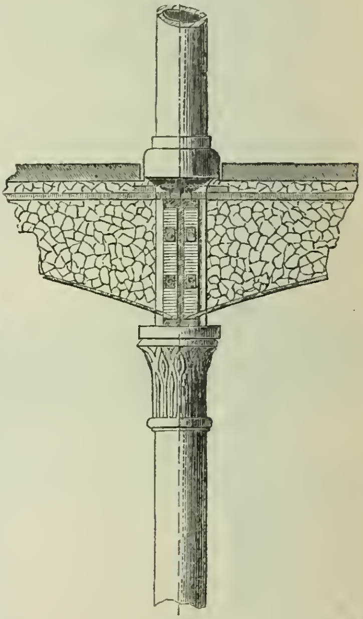


Fig. 69.



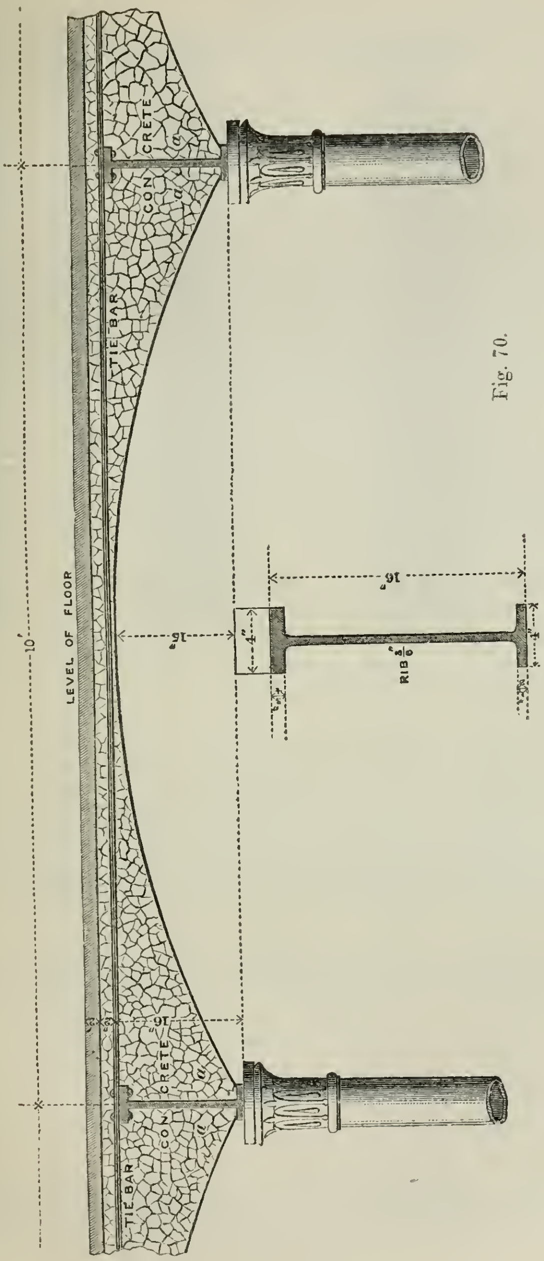
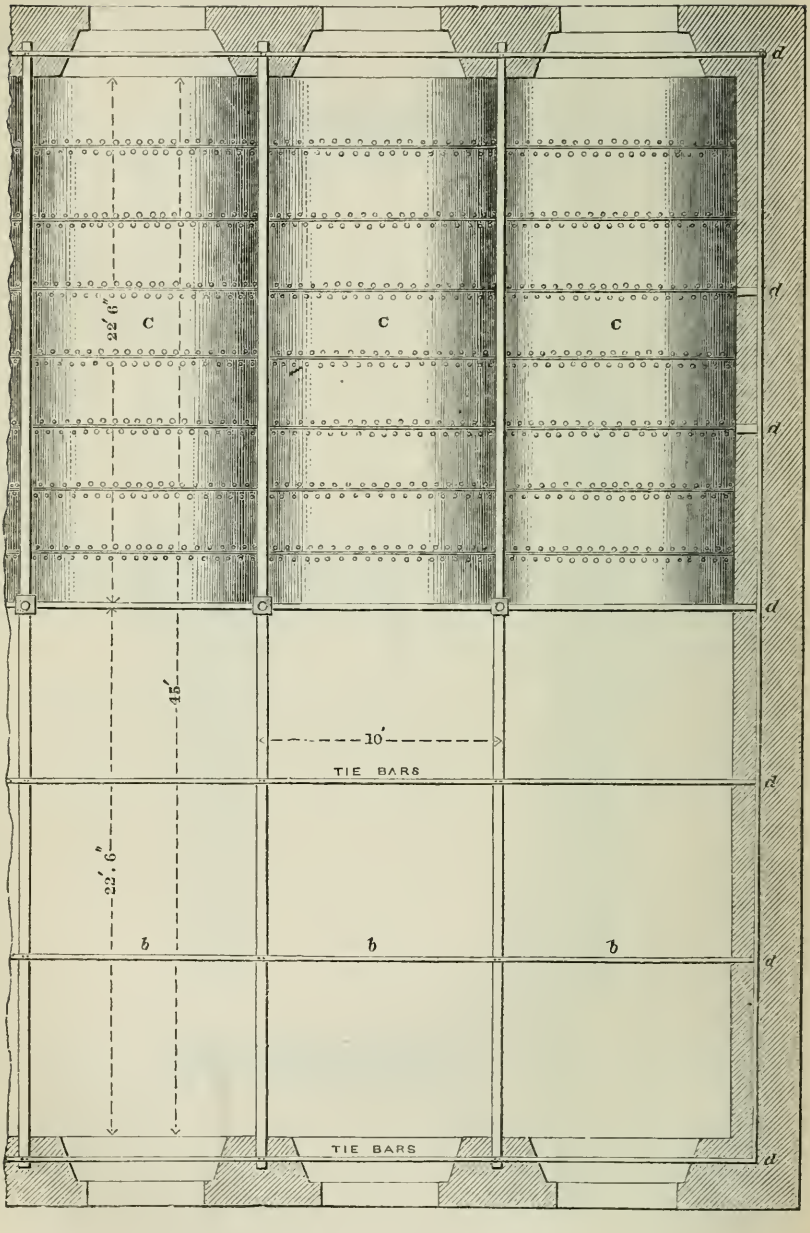


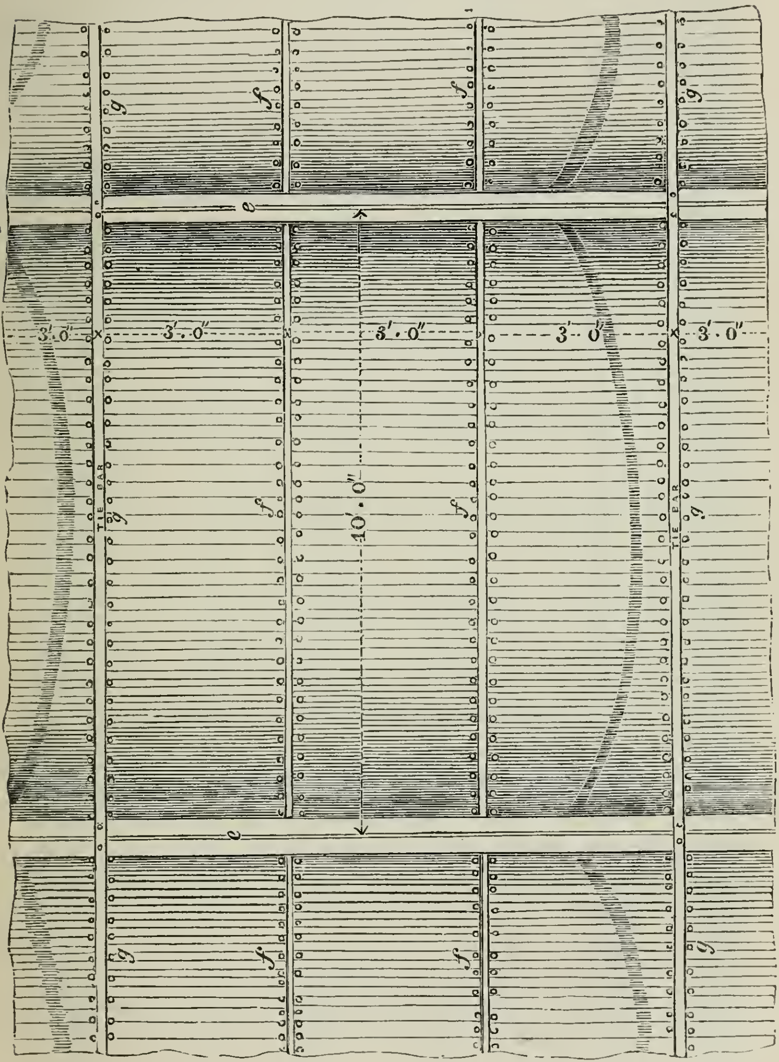
Fig. 70.

Fig. 71.



are built in the side-walls and gable-end of the mill, are riveted, as before stated, to the ends of the transverse beams which rest on the side-walls; and those along the gable are riveted to the ends of the rods, as shown at *d*, *d*, *d*, &c.

Fig. 72.



The connections and positions of the arch-plates are represented in plan at fig. 72, on a larger scale. It exhibits the method of uniting the plates by rivets to the **T** iron when looking down upon the floor, and also the tie-bars or rods which connect the whole of the transverse beams in each floor, from one extremity of the building to the other. In this case the tops of the beams are shown at *e*, *e*, the joints of the plates and **T** iron at *f*, *f*, *f*, &c., and the tie-rods at *g*, *g*, *g*.

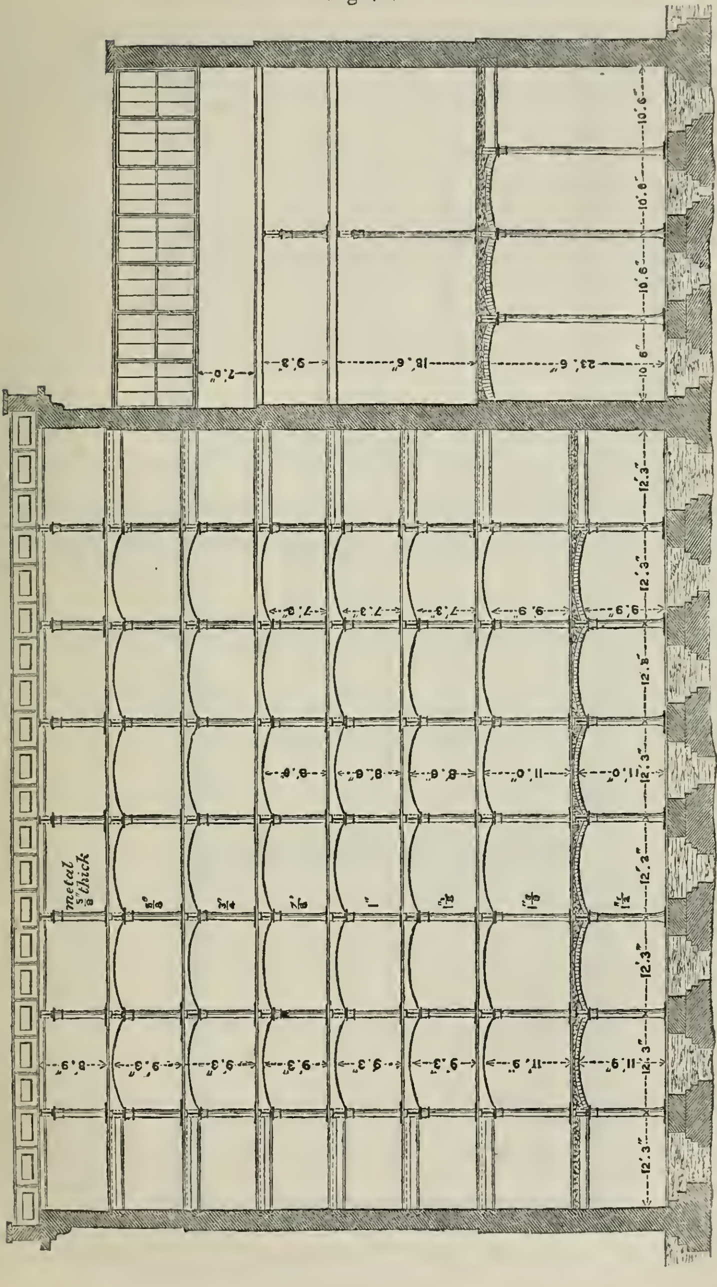
It will not be necessary to extend these inquiries further. The execution may be left to the skill and judgment of the builder. Every care and precaution should, however, be taken, in order to have secure foundations; and during the building to elevate the ends of the beams which rest upon the walls about half an inch in every floor, so as to allow for the settling of the walls, which from the superincumbent weight, generally settle more than the foundations of the columns.

Since the last edition of this work was published, I have had several opportunities of testing the value of wrought-iron beams; and in proof of their greater security and adaptation for building such as mills, warehouses, &c., where great weights have to be supported, I have selected a fire-proof building, eight stories high, erected for Messrs. Bewley, Moss, and Co., for illustration. It is built for a sugar refinery, and the weights which these floors and beams have to sustain, when loaded with moist sugar, has been calculated at 400 lbs. on the square foot, and the breaking weight of the beams is computed at 106 tons equally distributed.

This building is probably one of the most important yet constructed with arches in wrought-iron beams, and we may here refer to it as an example of what may be done by the introduction of a material free from flaws and much lighter than cast iron.

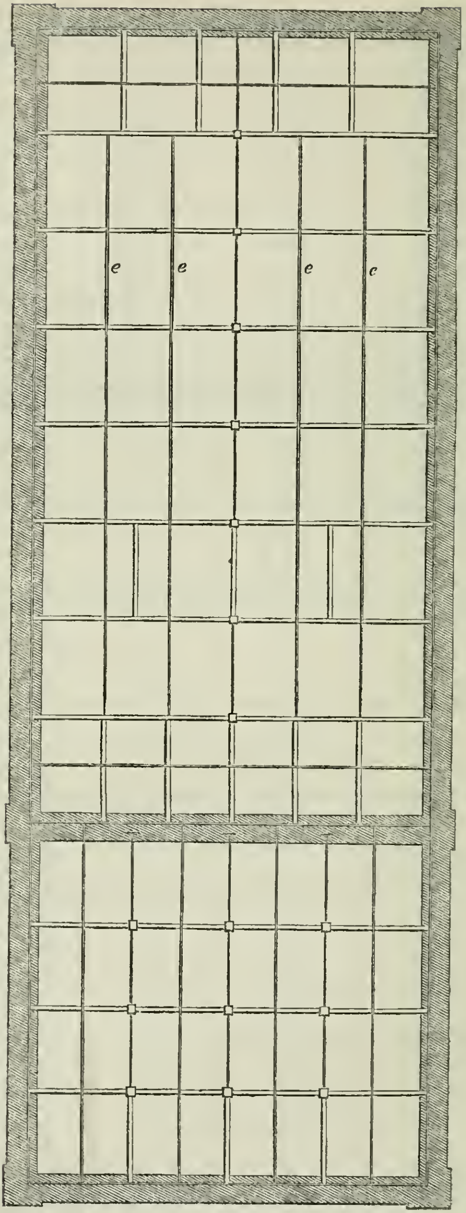
It will be observed that the building, figs. 73 and 74, is about 150 feet long, and 58 feet wide, with only one row of columns down the centre of the main building, and three rows in the lower floor of the cistern-house. Through the head of each column a 1 inch square tie-rod extends from one gable to the other, and is built into the arch, as shown at *a*, fig. 75. Transversely with the tie-rods are four bolts, *b*, *b*, &c., figs. 76 and 77,

Fig. 73.



Scale $\frac{1}{24}$ inch = 1 foot.

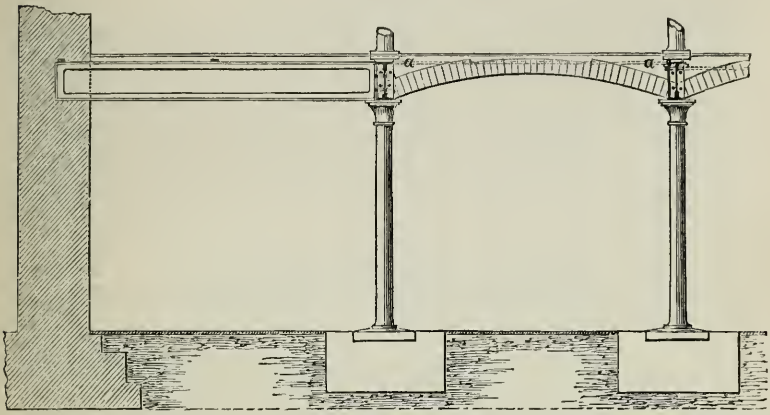
Fig. 74.



Scale $\frac{1}{24}$ inch = 1 foot.

which secure the ends of the beam to the columns. To the top flanges of the beams are riveted 6 flat tie-rods $2\frac{1}{2} \times \frac{3}{8}$, one built into the walls at each side, and the others equally divided between them and the centre of the building, as shown at *e, e, e,* &c., on the plan, fig. 74. These are to retain the beams straight in their places until the arches are turned from one end of the building to the other. From this it will be observed, that the tie-rods unite the arched floors and the walls on each side, forming an excellent bond, giving tenacity and unity to the structure *en masse.*

Fig. 75.



Scale $\frac{1}{8}$ inch = 1 foot.

Fig. 76.

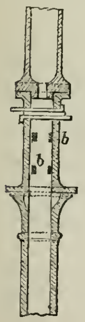
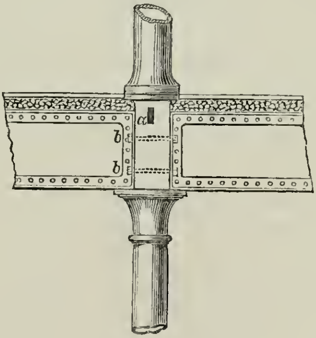


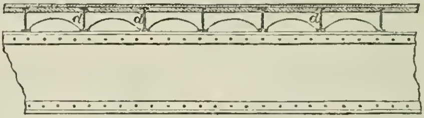
Fig. 77.



During the time of building or turning the arches, it is, however, necessary to be careful that, before the centres are struck, the thrust of the arch does not force the beams out of line by lateral pressure, as any undue force in that direction would prove highly injurious, and would seriously impair the strength of the beams. It is necessary to be very cautious in this respect, as wrought-iron beams, being more flexible than cast iron, are liable to yield to lateral pressure.

It will be seen, by the longitudinal section, fig. 73, that the brick arches are of the same construction as those for cast iron, but there is another mode of forming fire-proof floors with light wrought-iron joists resting on the main beams, as shown at *d*, *d*, *d*, &c., fig. 78. These joists are 6 inches deep and 15 inches apart; between each a large thin brick moulded

Fig. 78.



for the purpose is introduced. Over these is placed a covering of concrete or asphaltum, which forms a permanent fire-proof floor. If necessary, these could be covered with flags, tiles, or boards, according to the taste and requirements of the builder. I have made floors of this description with concrete alone, which answers every purpose when not subjected to rough usage, such as the rolling of loads or weights, or any other description of force tending to break the concrete.

Having given such description and detail as may be necessary for the guidance of the practical builder, I would, in conclusion, refer to the cotton mill of Messrs. John Whittaker and Brothers, near Ashton-under-Line, one of the most extensive structures of this kind, as illustrative of the arrangements and construction of these important edifices. The main building is 300 feet long, 60 feet wide inside, and six stories high. It contains, including the weaving-shed and warehouse, 24,000 square yards of flooring, about 40,000 spindles, and 1,800 looms. It cards, spins, and weaves into cloth about 54,000 yards, or 30 miles, of calico per day. In the transmissive machinery it gives motion to 92 or nearly 100 spur and bevel wheels, 4,800 feet of shafting, on which are fixed 1,250 drums or pulleys for driving the different machines; the whole of which weighs 460 tons. This great weight, exclusive of the preparatory and finishing machinery, is kept in motion at a velocity varying from 50 to 200 revolutions per minute, by two engines of the nominal power of 300 horses, but whose actual duty is equal to a force of upwards of 800 horses, in operation for 10 hours per day throughout the year. Taking the year at 316 working days, this gives the enormous production of 17,064,000 yards, or 9,480 miles of calico per annum.

I have given these statistics merely to show the extent and importance of this description of manufacture, and the extraordinary enterprise with which it is conducted. The establishment just referred to is in the possession of one family, three brothers; and besides this, they have another factory, which produces at the rate of 36,000 yards per day, and gives, as the aggregate producing power of the firm, an amount equal to nearly 16,000 miles of calico per annum.

Messrs. Whittaker's establishment is not the only one entitled to consideration on account of its powers of production; many others might be enumerated, equally extensive, if not more so, as regards both machinery and the facilities of execution; but few, if any, embody the same admirable system of concentration, and the same conveniences under one roof, for carrying on an extensive and productive manufacture. That of Messrs. John Fielden and Brothers, Todmorden, or that of Messrs. Thomas Ashton and Sons, Hyde, and many others, are immense establishments; but I have selected that of Messrs. Whittaker as the most appropriate for illustration, because it is the last mill, upon a large scale, which has been built in these districts, and embodies all the new improvements.*

It will not be necessary to enlarge further upon this topic, which is in some degree foreign to the objects of this inquiry, referring as it does to the buildings, and not to the machinery with which they are furnished. Suffice it to observe, that the construction of fire-proof buildings is equally applicable to every description of manufactory. I may safely state that the principle has been applied with considerable economy, and with indisputable success, to corn, cotton, flax, silk, and woollen mills, including every description of buildings for the manufacture of textile fabrics. It is also of great value in public buildings, such as barracks, prisons, workhouses, hospitals, granaries, and every other description which requires permanency of construction and security from fire. In private buildings, such as dwelling-houses, stables, outhouses, &c., it is equally efficacious; and from the exceedingly low rate at which iron is now manufactured, and the improvements that may hereafter be introduced, I entertain hopes that the time is not far distant when most of the buildings of this country shall be as secure from the corrosive effects of time as they will be from the ravages of fire.

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* The mills at Saltaire, near Bradford (see p. 177), have been erected since those of Messrs. Whittaker were built.

Having in this Treatise endeavoured to record the result of nearly thirty-five years' experience in these constructions, and having devoted much time, labour, and expense to the investigation of this important subject, I trust that the details which I have here given, as useful in my own practice, will be equally valuable to others for their guidance in constructions of such vast importance to the security of this description of property, either as regards durability or that measure of strength which constitutes the elements of their application.

It now only remains to direct attention to the mode of testing the bearing powers of cast-iron beams before they are used in building, and to give such directions as I have to offer on a subject which requires not only attentive consideration, but which, as a general rule, is not always clearly understood.

This is a subject on which there is considerable difference of opinion; and as I do not agree with several persons as to the extent to which beams should be proved, it will not, I apprehend, be thought irrelevant if I attempt to give my reasons for this dissent.

If we take into consideration those laws which govern the resistance of bodies under strain, or those physical truths which it is desirable that we should narrowly examine and closely follow, we shall arrive at conclusions of some value in the construction of buildings of this description. In pursuance of these laws, we must apply a principle calculated to test the powers, but not to injure the cohesive strength of the material under strain. This will appear obvious, as in all bodies subjected to a transverse or any other description of strain, there is a strong tendency in matter to resist disturbing causes; and this inherent antagonism in bodies we call strength or resistance to powers—however small they may be—which tend to disruption. Now, it is quite evident, that whenever a force is applied either to tear, break, or crush a body, be it iron, wood, stone, or any other solid material, it immediately calls into action its cohesive powers of resistance to that force. All bodies appear to have (if I may use the expression) a dread of disruption; and without inquiring into the laws which unite and bind their particles together, it is enough that we know the fact, and that we recognise in the varied powers by which this remarkable property—the force of cohesion—is maintained, a principle inherent in matter, and of which we are imperfectly acquainted, without knowing the cause, or in what manner these powers are prolonged, curtailed, or ultimately destroyed. These are parts of physical truth which should on no account be omitted in the education of professional men, as a more intimate acquaintance with these properties, or at least so far as they regard the use of material, and its varied application to the useful arts, would ensure greater safety to the public, and an increased variety of forms in symmetrical proportion.

The philosophy of the cohesive powers of bodies is, however, a perfectly distinct question. In this inquiry we deal exclusively with experimental facts; and all that we require to know is the ultimate powers of resistance offered by bodies to four kinds of strain, viz. tension, torsion, crushing, and a transverse strain. These being known, we can shape our course in every practical application with much greater certainty and effect than we could otherwise attain without this knowledge; and having once determined the powers of the material used, and the direction of the force applied, we can then with perfect security, and with the utmost certainty, determine the quantities necessary to be employed, and the position in which it should be placed, to ensure a maximum effect. These are a few of the general and leading principles which, it appears to me, constitute the security and economy of all those important structures which at the present day form so essential an element in the convenience and comfort of society.

A competent knowledge of a few leading principles which affect the strength of materials will enable us to determine the tests which it is requisite to employ in cast iron, in order to detect flaws and defects (if any exist) in the castings. It has already been observed, that cast iron should not be permanently loaded above one-third of the breaking weight. I believe that, in order to allow for contingencies, such as concussions from heavy weights falling upon beams, &c., it is safer to leave a larger margin, and never to exceed one-fourth of the breaking weight as a permanent load. In beams this is probably a necessary precaution; and in testing their powers and soundness when used for buildings, it is desirable, first of all, to load one of the beams, noticing the deflections with every increase of weight, till it breaks. This experiment is of great value, as it not only determines the comparative quality of the metal used, but exhibits its tenacity, elasticity, and other properties, which, in this age of experimental research, it is essential to determine. After the powers of the beam and the quality of the metal have been thus ascertained by the breakage of one or two beams, it will not be necessary to extend the proofs further than merely to lay on one-third the weight which broke the first beam, as a test for the remainder; and in laying on this weight care must be taken to ascertain the deflection and loss of elasticity or permanent set indicated by the state of the beam in comparison with that of the one broken, and the extent to which it is deflected by the weight and its removal respectively. All these changes should be recorded in a tabulated form in a book kept for the purpose. Such a book is a record of facts which is easily referred to, and from which the relative properties of each beam can at any time be determined. It might be desirable in some cases to test beams considerably above one-third of their breaking weights; but to these tests I have always had objections, as any severe strain is sure to disturb the molecular structure, and call into action a greater number of the component parts than is requisite in order to maintain the load. To this there is probably no serious objection, as the greater the number in action the less each will have to bear; but there is greater danger arising from unequal tension of the parts, which increased strain is apt to sever, whereby the ultimate strength of the beam may be seriously injured. I have known frequent injuries of this kind, which it is possible to detect by observing the deflections, and the consequent failure of the elastic powers, as evinced in the permanent set.

As an evidence of injuries done to beams by excessive tests, let us suppose that we load a beam, supported as at A, B, in the diagram, fig. 79, to within one-twentieth of the weight of rupture, say 950, and that 50 more would break it. From this it is obvious that nearly the whole of the resisting powers of the beam on both sides of the neutral axis are brought into play, and are suffering under severe pressure at the very point of rupture. Supposing, again, that we remove the load, and ascertain, by actual measurement, the effects of the tests to which it has been subjected, we shall then find that it has not only received considerable injury in its elastic powers, but is actually in danger of fracture from a repetition of the same load. Now, what is the result of a less severe strain? The beam, when loaded to one-third of the weight that was placed upon it, was in possession of powers that restored itself to within a fraction of its original position: whereas, with the heavy load it is seriously crippled, and a few more changes, even with reduced weights, would, from the derangement which the molecules or particles had sustained in the first instance, and the repeated alternations of removing and reloading, shortly destroy its powers of resistance, and cause fracture. It is for these reasons that I have always been opposed to severe proofs on cast-iron beams, or those of any other material; and I have no hesitation, in dealing with these useful constructions, in recommending a moderate test of about one-third, and that rather for the purpose of detecting flaws and imperfections in the casting, than for determining the ultimate strength of the beam.*

The resisting powers of beams (of whatever material they may be composed) are like the muscles of the animal frame when strained beyond their reasonable powers of resistance. They may for a time endure the load, and probably a few repetitions of it; but the result generally is, either the rupture of the several parts, or the total suspension of those powers by which their elasticity and powers of restoration are maintained.

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* The ultimate strength having been first ascertained by breaking one or two of the beams.

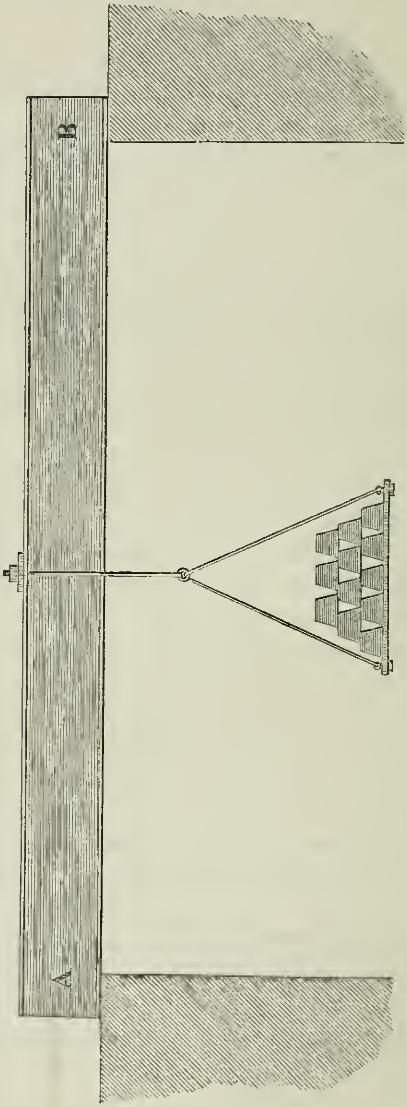

Fig. 79.

It therefore follows, that every description of material, when subjected to a transverse strain, should never be urged to greater endurance than may be required to straighten the fibres, or arrange the molecules of its crystalline structure. Any strain beyond that point is attended with risk; and in every case where the beam is subject to the alternate change of vibration, to dead weight, and the force of impact, it is safer not to allow the permanent load to exceed a fourth of the ultimate strength of the beam.

In girders for railway bridges, the permanent load should never be more than a fifth; and, in most cases, even a sixth is preferable, owing to the great weight and high velocities with which trains pass over a continuous line of rail, involving equally severe tests of impactive force on every structure, whether beams or bridges, that have to support the immense weight of railway traffic, varying in speed from 25 to 50 miles an hour.

Having in the preceding chapters or sections of this work endeavoured to place before the reader the results of the most recent investigations upon the different form of beams which enter largely into the construction of mills, warehouses, and other buildings, I may, with benefit to the professional practitioner, and as illustrating the preceding remarks, refer to an example where all these improvements have been brought into practical use. For this purpose, I select for illustration the gigantic establishment at Saltaire, near Bradford, Yorkshire; not more on account of its general completeness, than as a means of conveying to the mind of the general reader some idea of the vast energies, resources, and confidence which are brought to bear upon the development of manufacturing industry by the more advanced and enlightened men who are engaged in the production of textile fabrics. It is impossible to visit the neighbourhood of these busy hives, to survey the silent and uniform action of the great motive powers, to listen to the constant and confusing din of spindle and loom, to be informed of the number of human beings employed under one roof, of the amount of their earnings, and the astounding total of their produce; and to reflect further upon the enterprise and talent which must be in constant action, both abroad and nearer home, to keep this great whole supplied and at work,—without admiring the intellect which can guide such a work, and feeling thankful for that national security and prosperity which can justify the risk.

The Saltaire Mills are the property of Titus Salt, Esq., and are situated in one of the most beautiful parts of the romantic and well-known valley of the Aire. The site has been selected with uncommon judgment, as regards its fitness for the economical working of a great manufacturing establishment. The estate is bounded by highways and railways, which penetrate to the very centre of the buildings, and is intersected by both canal and river. Admirable water is obtained for the use of the steam engines, and for the different processes of manufacture. By the distance of the mills from the smoky and clouded atmosphere of a great town, an unobstructed and good light is secured; whilst, by both land and water, direct communication is gained for the importation of coal and all other raw produce on the one hand, and for the exportation and delivery of the manufactured goods on the other. Both porterage and cartage are entirely superseded; and every other circumstance which could tend to economise production has been carefully considered. The estate on which the town of Saltaire will gradually develope itself is very considerable in extent; and that part which is appropriated to the works, being literally covered with the buildings, is not less than six and a half acres. On this large surface of ground the heavy operations of the manufacture are carried on; but the superficies given to the several processes, and to the storage of goods, or, in other words, the floor-area of the establishment, is about twelve acres.

The main range of buildings, or the mill proper, runs from east to west,* nearly parallel with the lines of railway running from Shipley to Skipton and Lancaster. This pile is six stories high, 550 feet in length, 50 feet in width, and about 72 feet in height; and the architectural features, to avoid monotony in so large a dead surface, have been most skilfully treated by the architects, Messrs. Lockwood and Mawson, of Bradford. A bold Italian style has been adopted; and the beautiful quality of the stone of which the whole is massively built, displays the features of the structure to great advantage. Immediately behind the centre of the main mill, and at right angles with it, runs another six-story building, devoted to warehouse purposes, such as the reception and examination of the newly-manufactured goods; and on either side of this, again, lie the combing-shed or apartment wherein the fibres of the alpaca, mohair, wool, &c., are combed by machinery, the handsome range of buildings devoted to the reception of offices, and the great shed for weaving by power-looms. Of these features of the establishment it may be stated, that it was in the combing-shed that in September last 3,500 of Mr. Salt's guests sat down to dinner, without confusion or overcrowding, and with perfect ventilation; and that the great loom-shed would have similarly accommodated, under its single roof, more than double that number. Arranged in convenient situations are washing-rooms, packing-rooms, drying-rooms, and mechanics' shops, which will be more clearly understood by a reference to Plate II., which is a ground-plan of the whole mill at Saltaire. In the formation of the new roads which were requisite to secure free and easy access to the different parts of the mills, Mr. Salt was again not behindhand, and availed himself of the most recent experience of scientific practice; therefore we find bridges of the most solid and durable construction both in cast and wrought iron: one of these viaducts, on the tubular-girder system, crossing the canal and river Aire, being not less than 450 feet in length.

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* See Plates I., II., and III.

With this brief recital of the general distinguishing characteristics of Mr. Salt's mill, I will now enter more into detail on those points of interest which bear more directly upon the subject of this work.

1. *Construction.*—The mill marked A, Plate II., has a cellar-story 10 feet high, extending on either side of the engine-houses to the extreme ends of the mill. The first story is 16 feet from floor to floor; the next four stories are each 14 feet high; and the fifth or attic-story measures 8 feet from the floor to the tie-rod (Plate III.), and a further distance of 9 feet 6 inches from the tie-rod to the apex of the roof, making the total height of this attic-story 17 feet 6 inches. It must be further remarked, that this top story, being above the levels required for the entrance-archway and the two engine-houses, extends from end to end of the building, 550 feet; and is

supposed to be the largest room in Europe, not excepting the Long Gallery of the Louvre at Paris. The whole of this building is fire-proof, with stone walls, cast-iron columns, and hollow brick arches. By referring to the cross section of the mill, given in Plate III., it will be noticed, that the side-walls throughout the entire length of the structure are made hollow, as shown at *a*, *a*, *a*, *a*, &c. Through these openings a supply of pure fresh air to each room is admitted and regulated a few inches above the floors; and corresponding outlets for the heated or vitiated air being provided above, a simple but most efficient ventilation is obtained, and at all times and seasons an adequate circulation of air is passing through the mill. Each room is longitudinally divided by a single row of columns; and it was found expedient to form two unequal spaces in order to accommodate the machinery, and leave in the centre a thoroughfare free from obstruction. The divisions are, therefore, respectively, 27 feet 6 inches and 22 feet 10 inches. In its construction as a fire-proof structure, the mill varies from the usual practice in one important particular, the arches for supporting the floors being constructed of hollow bricks, the form and section of which are accurately shown in figs. H and I, Plate III. This adaptation is a great improvement, inasmuch as it greatly diminishes the dead weight and strain upon the iron beams; and the bricks themselves having been moulded to the proper curve, an even surface is obtained for the soffit or under side of the arch. This work then simply required pointing, and the plastering of the arches has been entirely dispensed with. The sectional drawing, K, shows the span and versed sine of the arches, the haunches, *b*, *b*, *b*, being filled with concrete and levelled, to give a solid and equable bearing for the floors, which are made of Yorkshire flag-stones, 2¼ inches thick. The cast-iron beams are of the section of greatest strength, as shown in the following tables. For the wider span of 27 feet 6 inches, they were made 18 inches deep in the middle, and 11 inches deep at the ends; and for the smaller span of 22 feet 10 inches, 17 inches deep in the middle, and 10 inches at the ends. The mixture of metals from which they were cast was as follows:—

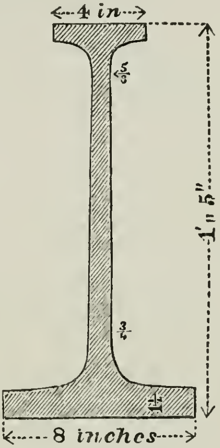
Gartsherrie, No. 3. }
Dundyvan, No. 3. } Equal parts of each.
Hematite, No. 3. }

And by computation, the power of the two beams to resist a transverse strain in the middle was found to be 29½ tons and 22¼ tons for the larger and smaller ones respectively. In order, however, to test the quality of the metal, a casting from each beam was fractured by the suspension of a dead weight of pig-iron from the centre. The following are the results of these experiments:—

EXPERIMENTAL TEST OF BEAMS 22 FEET 6 INCHES BETWEEN THE SUPPORTS.

Tons.	cwt.		Deflection in inches.
7	0	on the centre . .	·31
7	18	” ” . .	·43
8	15	” ” . .	·46
9	12	” ” . .	·56
10	9	” ” . .	·62
11	6	” ” . .	·65
12	3	” ” . .	·75
13	0	” ” . .	·81
13	17	” ” . .	·84
14	14	” ” . .	·90
15	11	” ” . .	·92
16	8	” ” . .	1·06
17	5	” ” . .	1·12
18	2	” ” . .	1·18
18	19	” ” . .	1·25
19	16	” ” . .	1·31
20	13	” ” . .	1·43
21	10	” ” . .	1·50
22	7	” ” . .	Broke with this weight.

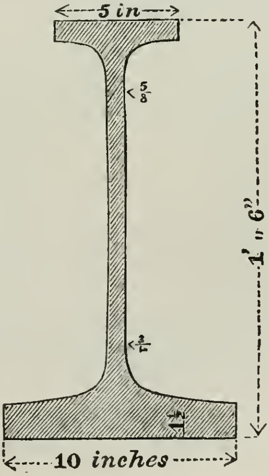
Fig. 80.



EXPERIMENTAL TEST OF BEAMS 27 FEET 4 INCHES BETWEEN THE SUPPORTS.

Tons.	cwt.		Deflection in inches.
8	15	on the centre . .	·50
9	12	” ” . .	·56
10	9	” ” . .	·59
11	6	” ” . .	·65
12	3	” ” . .	·75
13	0	” ” . .	·84
13	17	” ” . .	·93
14	14	” ” . .	1·06
15	11	” ” . .	1·12
16	8	” ” . .	1·18
17	5	” ” . .	1·31
18	2	” ” . .	1·43
18	19	” ” . .	1·56
19	16	” ” . .	1·71
20	13	” ” . .	1·84
21	6	” ” . .	1·96
21	16	” ” . .	2·12
29	6	” ” . .	On account of the danger of approaching too near the beam, the deflections, till 29½ tons were laid on, were not taken. The beam ultimately broke with that weight.

Fig. 81.



About 200 of the long beams having been tested with 12 tons in the centre, they deflected differently from $\frac{3}{8}$ to $\frac{5}{8}$ of an inch.

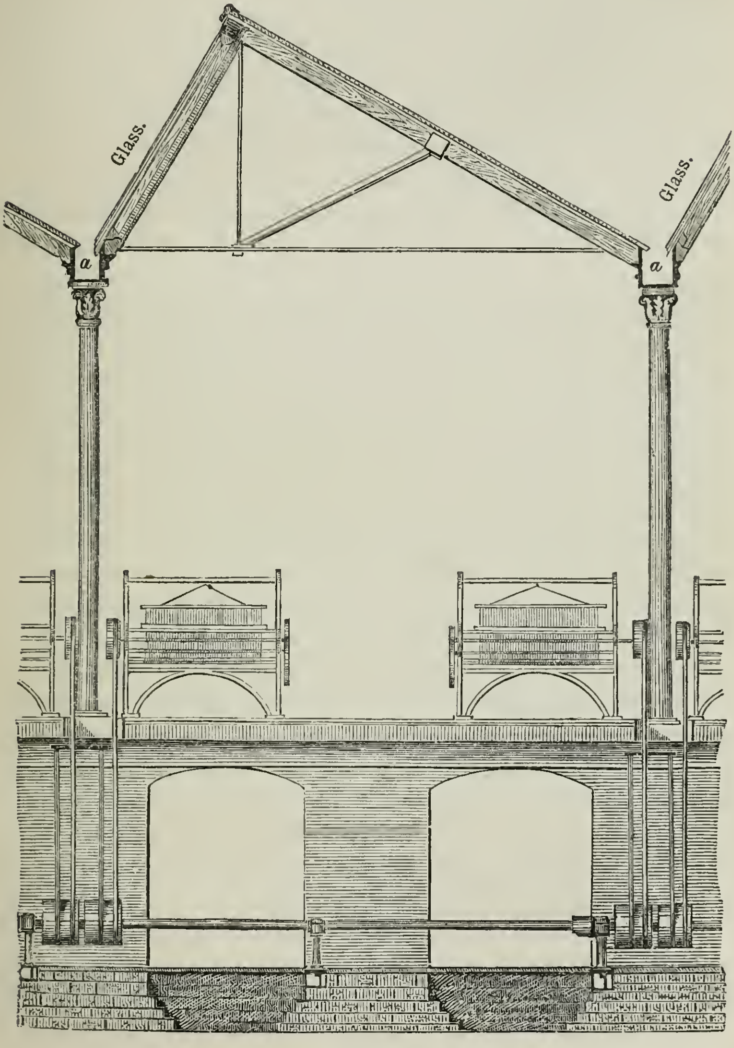
The same number of short beams, tested with 8 tons in the centre, deflected differently from $\frac{7}{16}$ to $\frac{8}{16}$ of an inch.

From the above tests it will be seen that the mixture or compound of the three irons indicated a strong and superior quality of metal.

The roof, like the other parts of the building, is entirely fireproof. It is composed of 128 principals, formed of T-iron, angle-iron, and tie-rods, as exhibited in section at G, Plate III. The ends of the principals are fitted into cast-iron shoes, which rest upon the walls; and to those again are attached the horizontal transverse rods, G, which support the principals, and retain the whole in form. From the top, on each side of the ridge, and along the whole length of the building, the room is lighted with glass, and the position of the frames is so situated as to render the room at once light and cheerful in appearance. In other respects, the only objectionable feature, in such a long and extended room, is the comparative lowness of the roof; but when it is considered that the structure is made for the purpose of utility, and not for exhibition, the impression vanishes, and the conception of the objects to be attained becomes more distinct and more strikingly apparent. Besides, it must be borne in mind, that any enlargement of this part of the edifice would have increased the expense of maintaining uniformity of temperature; and that, more particularly in a situation so much exposed to atmospheric influence, would have proved detrimental, if not seriously injurious, to the manufacturing process, and the machinery employed in the room.

The large weaving-shed, marked C, Plate II., on the plan, it will be observed, is divided into parallelograms of 36 feet in one direction, and 18 feet in the other. At the angles of each division cast-iron columns are fixed, which support 13 lines of cast-iron gutters, marked *a*, *a*, fig. 82; and these gutters are constructed of such dimensions as to form entablatures for the columns, and supports for the reception of the roofs, which extend from east to west, in the same direction; as also the glass divisions in every compartment which face the north, and give nearly at all times of the day a steady uniformity of light. The combing-shed, D, is of a similar construction; but with this exception, that it is divided into squares of 18 feet, and supported by columns and roofs in the same way as those of the weaving-shed. The annexed woodcut, fig. 82, gives a section of

Fig. 82.



this part of the construction, showing the position of the columns, the angle of the lights, and a portion of the cellar, which contains the driving shafts, wheels, pulleys, &c., that give motion to the machinery, to the amount of 1,000 looms or upwards, in the room above.

The peculiar features in this department are, that the mill-work and driving-gear are all below the floor; and the whole of the immense area is free from the obstruction of straps, wheels, drums, or other impediment, tending to obstruct the view. The result of this is, that the overlooker has full command of the whole room at a single glance.

2. *Gas and Water Supply.*—In these necessary adjuncts to an establishment of such magnitude, every precaution has been taken to insure a regular and constant supply. The gas works are situated at the lowest point of the land, between the canal and the river; and the gasometer tank, 62 feet diameter, and 20 feet deep, is sufficiently low to be filled either from the large water-pipes or reservoir intended to supply the town of Saltaire, or from the canal, which flows at a level of about 23 feet above the bed of the river. The gas-holder is 60 feet in diameter, 18 feet deep, and contains 50,000 cubic feet of gas, or a supply sufficient to maintain 5,000 lights. The works are supplied with soft rain-water collected from the roofs, and the condensed water from the steam used in heating the mill. These waters are conveyed by pipes into a large tank or cistern under the combing-shed, where it is preserved and filtered for the cleansing and washing of wool. For the supply of water to the town for domestic use, a well is sunk to a considerable depth under the foundation of the steam engines; and a 9-inch pump is attached to one of them, for the exclusive purpose of raising the pure spring water into a reservoir of sufficient altitude to command the whole of the town, on the gravitating or high-pressure principle.

3. *Heating and Ventilation.*—To maintain a uniform temperature, and to change the atmospheric contents of the different rooms where numbers of persons are employed, has been a question of anxious solicitude on the part of the architect and engineer, as well as Mr. Salt; every known precaution has been taken, and every known improvement

adopted, to secure an agreeable temperature and a healthy ventilation. It has already been stated, that the walls of the main building have been perforated for the admission of pure, and the discharge of the saturated air; and in addition to these provisions, each room has a double row of steam-pipes, heating the upper strata of air to a temperature of 60°, and thus causing a constant circulation of imperceptible currents to be passing through the rooms.

The weaving-sheds, which are lofty and more exposed, are heated and ventilated by a different process. The system is here accomplished by a mechanical operation of forcing air through two or more cylinders, filled with small tubes. These tubes are kept heated by the exhaust-steam of two non-condensing high-pressure engines, employed to drive the fans by which the air thus heated is forced along channels under the floor, and passing along between the groins of the arches, is ultimately discharged into the room by register-valves, at numerous inlets from the channels below. This apparatus, the invention of Messrs. Hamilton and Weems, of Johnstone, near Glasgow, is expected to maintain a constant circulation of warm air into the room throughout the winter, and cold air during the more intense heat of the summer months.

4. *Motive-Power, Mill-Work, and Machinery.*—The prime-movers of the Saltaire mills are two separate pairs of condensing steam engines, situated, as shown on the ground-plan, at each side of the main entrance. These engines are calculated to exert collectively a force of not less than 1,250 horses, calculated according to the usual rule of a horse-power, being equal to a weight of 33,000 lbs. raised one foot high in a minute. They are massive and imposing in design, and are proportioned in their parts to work with that great economiser of fuel, high-pressure steam. The steam is generated in ten multitubular boilers, placed in front of the mill, but below the level of the ground. Their site is indicated by the dotted lines on the ground-plan, Plate II. The boilers are constructed with a view to the prevention of smoke, and the least possible consumption of coal. For these purposes, every precaution has been taken to prevent the escape of heat: the boilers are carefully covered with non-conducting material; and even the

steam-pipes for the supply of the engines, after leaving the boilers, are carried into the main flue, where the steam is relieved of all vapoury particles it may have carried with it, and, surcharged with heat, enters the cylinder for the exertion of its power in a highly elastic and effective state. The flame and gases from the boiler-furnaces are gathered into one main flue, which runs direct to the chimney-shaft. This chimney, which, under the skilful treatment of the architects, is an ornament and finish to the general mass of the buildings, instead of an eyesore, stands at some little distance in front, and rises majestically to a height of 250 feet. It is built of stone, in the style of an Italian campanile; but to protect the column from the action of the heated currents, and to preserve a parallelism of flue from bottom to top, it is lined with a casing of fire-brick, which leaves a space filled with warm air protecting the external walls, and maintaining a perfectly uniform and efficient draught.

The two pairs of engines work independently of each other; but in both cases the power is taken direct from the peripheries of the fly-wheels, the recipients of their motion being spur-wheels, geared with wooden teeth. The requisite motion and speed is carried from the main wheel to the remotest part of the establishment, by shafts and wheels in the usual way. The total length of shafting is upwards of 10,000 feet, or very nearly 2 miles, varying from 14 inches to 2 inches diameter, weighing upwards of 600 tons, and revolving at velocities varying from 60 to 250 revolutions per minute. It will not be necessary to describe in detail the different classes of machinery; suffice it to say, that Mr. Salt, with that forethought and liberality which are absolutely essential to the successful and profitable working of all such concerns, determined that it should be new throughout, and that it should embody the results of the newest invention and the most perfect construction.

The capabilities of Saltaire mills may be possibly measured by the statement, that it will contain within itself the means of every kind of preparing from the raw material, the supply for not less than 1,200 power-looms; and that the daily yield or production of those 1,200 looms is the astounding total of 30,000 yards of alpaca, or mixed cloth, per day, or upwards of 5,000 miles per annum.

PART IV.

THE ADAPTATION OF MALLEABLE-IRON BEAMS OR GIRDERS FOR THE CONSTRUCTION OF BRIDGES.

Bridges have been in use as a means of facilitating conveyance from a remote antiquity, and have received in all ages that consideration to which, from their great importance and public utility, they are so justly entitled. They form connecting links between separate lands,—they cross the rapid river and the deep ravine, and fling a safe roadway over places otherwise impassable or difficult of access. They are among the most valuable means possessed by the civilised world for facilitating that interchange of natural and manufactured products, from which arises the intelligence and prosperity of nations. To them, in some measure, we owe all the benefits we now derive from steam communication; for without them, the vast railway system, which is spreading its network over every land, could not exist. Surely, then, every improvement and every new application which increases our power of contending with the irregularities of the earth's surface, is entitled to the most careful consideration of the philosopher, the engineer, and the community at large.

Passing briefly in review the principal forms of bridges now in use, we find that nearly all those of a fixed or permanent character, though of great variety in appearance and construction, are readily resolved into three classes, sufficiently convenient and accurate for the objects contemplated in this work:—

1st. There are those like the stone arch, supported by resistance to compression;

2nd. There are others called suspension bridges, supported by resistance to tension; and,

Lastly, the remainder are supported by combined resistance to tension and compression, and are known as beam and girder bridges.

History does not inform us when these forms of bridges were first introduced, but we may reasonably conclude that they must have been coeval with the earliest attempts at construction, and that it could not have escaped the notice of the first workers in stone, iron, and wood. I conceive that the intelligent race who invented the plough, the loom, and instruments of music, would never allow a construction so palpably useful to elude their notice. The earliest bridges were probably of timber, and simply consisted of the trunks of trees thrown across rivers from bank to bank. Stone might also have been employed in some cases on the system of corballing, by placing quarried blocks upon one another on either side, with their ends projecting one over the other, and continuing the process till they met in the centre. This was done by the ancient Egyptians before the arch was discovered, as may be seen in some of the architectural remains of that interesting people.

These and other erections were probably in use ages before the time of Xerxes, who constructed the bridge of boats across the Dardanelles—a great achievement at that time, though surpassed by Trajan's Bridge across the Danube and Cæsar's across the Rhine. Great as were these undertakings, even they are probably inferior to some ancient structures of the Chinese; and, indeed, it may be questioned whether the most stupendous works of modern times, the bridge of Telford over the Menai Straits, or that of Freiburgh in Switzerland, when their respective dates are considered, are superior in boldness of design and execution to some of these masterpieces of the ancient Chinese.

The increase of commerce in modern times has, progressively, led to the erection of bridges of a more durable character and larger dimensions than those of ancient date; and down to comparatively recent times, the stone arch has maintained a decided superiority over the more perishable structures; whilst as better modes of construction were discovered, and greater minds employed in their design and erection, their spans have been enlarged and their beauty increased, till they have taken rank among the noblest results of architectural and engineering art.

The immense extension of the railway system in this and other countries has necessitated a great number of new constructions, such as tunnels, cuttings, embankments, &c.; and among these, bridges hold a prominent place in utility and importance.

The strength, security, and symmetrical proportions of bridges have ever attracted the attention of the most eminent architects and mathematicians: and at a time when new discoveries in chemistry, electricity, and mechanics, are causing a revolution in every branch of science and art, it is not surprising that bridges of new capabilities and of improved design should be brought into use. At no period of history has the demand for bridges been so large, or the difficulties to be encountered so great, as they have been since the introduction of railways,—and yet, never has that demand been so cheerfully met, and never have the difficulties been so energetically and so effectually overcome, as during the last thirty years.

The innumerable number of interests that have to be consulted, and the various character of the obstacles to be surmounted, in the construction of railways, have led to the erection of bridges of almost every description. Sometimes different constructions have been adopted from necessity, at others from caprice, and in many instances the skew arch is chosen, not from any inherent merit of its own, but because it appears more ingenious and more difficult of construction than the simple rectangular bridge.

Of these various forms, we may notice the cast-iron skew and rectangular arches for wide spans; the bowstring bridge, with an arch of cast iron above and strong tension rods below, as chords to the arch, the roadway being suspended from the arch above by vertical rods; the wooden laminated arch of American origin, which has the roadway either supported above by framework rising from the top of the arch or suspended below, as circumstances require: to these we may add the trussed timber and trussed cast-iron bridges, and many others which have been put in requisition, and more or less advocated for various good qualities, such as compactness, economy, or strength.

There exists great difference of opinion as to the comparative merits of these various forms in their applicability to the requirements of road and railway traffic, but on one point most of the profession are agreed, viz.:—the peculiar advantages of the solid horizontal form. On all sides it seems to be allowed that a bridge, having a perfectly horizontal soffit, united to great strength, compactness, and durability, is a desideratum; more especially where it is necessary to have the roadway level, and to obtain the greatest possible height under the bridge. These conditions are almost always required in the case of railways crossing navigable rivers, canals, or roads; and to supply these requirements the horizontal tubular girder and plate girder bridges have been employed, as not only the most economical, but as being the strongest and most rigid in spans varying from 40 to 400 feet.

Since the introduction of the tubular system and the completion of the stupendous bridges over the Conway and Menai Straits, many attempts have been made to vary the form, so as to produce a bridge of an apparently new construction; but almost all horizontal bridges are built on the principle of the beam or girder, which is sustained by its resistance to the forces of tension below and compression above. Even the lattice and bowstring bridge, and the compound bridge of Mr. Brunel, as erected at Chepstow and Saltash, although very different in appearance, partake in almost every respect of this principle of construction.

Before we proceed to consider the great variety of bridges which present themselves, it will be necessary to devote some attention to the materials, such as timber, stone, brick, iron, &c., of which they are composed. In the construction of bridges circumstances vary, and often dictate the material to be employed. 'The locality,' as observed by Mr. Hosking, 'will often point out the material by furnishing one particular kind and denying others, and the use of that kind economy will very generally impose;' but it still remains for consideration whether or not some other material than that which the locality supplies may not be adopted for reasons of economy and expediency to meet the requirements of traffic, and to insure the durability of the structure.

Timber.

Of all the materials employed for bridge building, timber is probably the most extensively and variously available; it is the material by the use of which the greatest amount of work can, in most cases, be effected at the least cost, and in the shortest time; whilst in most countries it affords greater facilities for construction than any other description of material. Its power of resistance to strain depends on its quality, as is shown in the following tables, which exhibit its resistance to tension and compression, which it will be observed is much inferior to stone and iron. The following table gives the result of some experiments carefully conducted by Mr. Hodgkinson at my works in 1839.

The specimens on which these trials were made, were turned into cylinders of about one inch in diameter and two inches long; the crushing surfaces were perfectly parallel, and the specimen to be crushed had its ends firmly bedded against them; the power being applied in the direction of the fibres. The specimens broke by sliding off at a constant angle, dependent on the nature of the material. And this Mr. Hodgkinson found to be the case in his experiments on cast iron and other bodies; showing that the strength in any particular species of bodies varies directly as the area of the section. Great discrepancies occurred when the woods were of different degrees of seasoning

Compressive Strength of Wood.

Description of Wood.	No. of Experiments.	Mean force per square inch required to crush the specimen.
Yellow pine	3	5,375
Cedar	3	5,674
Red deal	3	5,748
Poplar (not quite dry)	3	3,107
Poplar (dried two months, length 1 inch)	1	5,124
Larch (green)	3	3,201
Larch (dried one month)	1	5,568
Plum-tree (wet, though cut two years)	3	3,654
Plum-tree (dried two months)	3	8,241
Birch (green)	3	3,297
Birch (dried two months, 1 inch long)	3	6,402
Sycamore	3	7,082
Ash	3	8,683
Ash (dried two months, length 1 inch)	1	9,363
English oak	3	6,484
English oak (dried two months, length 8 inches)	2	9,509
Spanish mahogany	3	8,198
Box-tree	7	9,771

and dryness; wet timber, though felled for a considerable time, bearing in some instances less than half what it would have borne when dry.

With the exception of the poplar, larch, birch, plum, and oak, the above specimens were moderately dry; and where it is stated that they were kept for any particular time, it is to be understood that they were prepared specimens, which not being used at the time the earlier experiments were made, were kept in a warm dry place during the time mentioned, and then remeasured and crushed.

If we now take the strength of direct cohesion from Muschenbroeck and Barlow, we shall have for their resistance to a tensile strain the following proportions.

TENSILE STRENGTH OF WOOD—ACCORDING TO MUSCHENBROECK.

Description of Wood.	Tensile strength per square inch in lbs.	Description of Wood.	Tensile strength per square inch in lbs.
Locust-tree	20,100	Pomegranate . . .	9,750
Locust-tree	18,500	Lemon	9,250
Beech and oak . .	17,300	Tamarind . . .	8,750
Orange	15,500	Fir	8,330
Alder	13,900	Walnut	8,130
Elm	13,200	Pitch-pine . . .	7,630
Mulberry . . .	12,500	Quince	6,750
Willow	12,500	Cypress	6,000
Ash	12,500	Poplar	5,500
Plum	11,800	Cedar	4,880
Elder	10,000		

ACCORDING TO BARLOW.

Description of Wood.	Tensile strength per square inch in lbs.	Description of Wood.	Tensile strength per square inch in lbs.
Box	20,000	Beech	11,500
Ash	17,000	Oak	10,000
Teak	15,000	Pear	9,800
Fir	12,000	Mahogany . . .	8,000

Comparing the above experiments on tensile strength with those of Mr. Hodgkinson on the resistance opposed to a crushing force, we deduce the following comparative ratios of strength.

Description of Wood.	Mean crushing force per square inch.	Mean tensile strength per square inch. Muschenbroeck.	Mean tensile strength per square inch. Barlow.	Ratio: crushing force = 1·00.
Box	9,771	—	20,000	1 : 2·04
Ash	8,683	12,500	17,000	1 : 1·69
Fir	5,748	8,330	12,000	1 : 1·76
Oak	7,996	17,300	10,000	1 : 1·70
Plum-tree	5,947	11,800	—	1 : 1·98
Mahogany	8,198	—	8,000	1 : 0·97

Exclusively of timber being applicable as a material of which bridges may be built, it is an essential auxiliary in the erection of bridges of any other material. But although possessing so many advantages it is of all building materials the most liable to decay, and is most exposed to injury if not entire destruction from fire. It is also subject to change during seasoning from the evaporation of its sap; and the constant shrinkings and swellings occasioned by changes in the hygrometric state of the atmosphere are sure indications of rapid decay. Hence a timber bridge is objectionable both as regards durability and ultimate economy. Various expedients have been resorted to in order to prevent atmospheric changes influencing timber; but few of them afford the necessary protection in a climate so variable as that of the British Isles.

Stone.

Stone is admirably adapted for bridge-building, on account of its unyielding nature and its great power of resistance to compression. It may be cut to any form, and it is, practically speaking, uninfluenced by atmospheric changes, by heat, or by those causes which so greatly affect substances absorbent of water, of high conducting power or of much affinity for oxygen. These properties render it applicable in most situations where it can be obtained in large quantities and at moderate cost. It cannot, however, be employed where great spans are necessary, as we know of no instance where a stone bridge exceeds a span of 250 feet.* Hence in all large structures it becomes a question whether some other material may not be advantageously used in order to save the enormous expense of stone arches of large span.

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* The Chester Bridge, built by Mr. Harrison, has a span of 200 feet. That which formerly stood at Trezzo, over the Adda, had an arch of 251 feet span: the versed sine of the segment rather exceeding one-third of the chord.

As all stone bridges are supported exclusively by *compression*, and since stone is rarely used otherwise, it will be sufficient to investigate its powers of resistance to this force, in order to establish those rules which in practice should guide the engineer in the selection and application of the material. As these rules are derived from direct experiments on a great variety of specimens, we may rely with greater confidence on the results.

In the construction of the Conway and Britannia bridges fears were entertained of the security of the masonry of the piers under the enormous pressure of the superincumbent tubes of iron, and hence it was considered necessary to ascertain the resisting powers of the materials employed in their construction. The following are the results of some experiments conducted by Mr. Latimer Clarke, January, 1848.

Sandstone.

	Lbs. per square inch.
1.—3 in. cube of red sandstone, weighing 1 lb. $14\frac{3}{8}$ oz., set between boards (made quite dry by being kept in an inhabited room). Crushed with 8 tons 4 cwt. 0 qrs. 19 lbs.	=2,043
2.—3 in. cube sandstone, weighing 1 lb. 14 oz., set in cement (moderately damp). Crushed with 5 tons 3 cwt. 1 qr. 1 lb.	=1,285
3.—3 in. cube sandstone, weighing 1 lb. $15\frac{1}{2}$ oz., set in cement (made very wet). Crushed with 4 tons 7 cwt. 0 qrs. 21 lbs.	=1,085
4.—6 in. cube sandstone, weighing 18 lbs., set in cement. Crushed with 63 tons 1 cwt. 2 qrs. 6 lbs. . .	=3,924·8
5.—9 in. cube sandstone, weighing $58\frac{1}{2}$ lbs., set in cement. ($77\frac{1}{2}$ tons were placed upon this without effect, = 2,042 lbs. per square inch, which was as much as the machine would carry).	
Mean	=2,185

All the sandstones gave way *suddenly*, without any previous cracking or warning. The 3-inch cubes appeared of ordinary description; the 6-inch was fine grained, and appeared tough and of superior quality. After fracture the upper part generally retained the form of an inverted pyramid about $2\frac{1}{2}$ inches high and very symmetrical, the sides bulging away in pieces all round. The average weight of this material was 130 lbs. 10 oz. per cube foot, or 17 feet per ton.

The average weight required to crush this sandstone is 134 tons per square foot, equal to a column 2,351 feet high of such sandstone.

Limestone.

Lbs. per square inch.

1.—3 in. cube Anglesea limestone, weighing 2 lbs. 10 oz., set between boards. Crushed with 26 tons, 11 cwt. 3 qrs. 9 lbs. . . . . =6,618

This stone formed numerous cracks and splinters all round, and was considered crushed, but on removing the weight, about two-thirds of its area were found uninjured.

2.—3 in. cube limestone, weighing 2 lbs. 9 oz., set between deal boards. Crushed with 32 tons 6 cwt. 0 qrs. 1 lb.=8,039

This stone began to split externally with 25 tons (or 6,220 lbs. per square inch), but ultimately bore as above.

3.—3 in. cube limestone, weighing 2 lbs. 9 oz., set in deal boards. Crushed with 30 tons 18 cwt. 3 qrs. 24 lbs.=7,702·6

4.—Three separate 1 in. cubes limestone, arranged in a triangle, weighing 4½ oz., set in deal boards. Crushed with 9 tons 7 cwt. 1 qr. 14 lbs. . . . . =6,995·3

All crushed simultaneously.

Mean . . . . =7,579

All the limestones formed perpendicular cracks and splinters a considerable time before they crushed.

Weight of material from above =165 lbs. 5 oz. per cubic foot, or 13½ feet per ton.

The weight required to crush this limestone is 471·15 tons per square foot, equal to a column 6,433 feet high of such material.

This was the material employed for the towers of the Menai tubular bridge, and the experiments were commenced in order to ascertain whether this description of stone was capable of supporting the enormous weight of the iron-work contained in the tubes. Subsequently, it was deemed expedient to build into the piers cast-iron beams, covering a large surface upon which the ends of the tubes might rest. By this means the pressure was distributed over a large area, and all risk of danger avoided.

To the above experiments may be added some others recently conducted by the author on a great variety of stone from different parts of the kingdom.* The specimens subjected to experiment were smaller than those employed for the tubular bridges, being only 1, 1½, and 2 inch cubes, but the breaking weights were carefully observed, and recorded as follows:—

EXPERIMENTS TO DETERMINE THE FORCE REQUIRED TO CRUSH DIFFERENT KINDS OF STONE.

No. of Specimen.	Description of stone.	Locality.	Size of cube. in.	Specific gravity.	Pressure to fracture specimen.	Pressure to crush specimen.	Pressure per square inch to crush specimen.	Cubic feet in a ton.
1	Sandstone	Shipley†	2	2·452	33,524	38,900	9,725	14·616
2	,,	Heaton†	,,	2·420	33,524	40,692	10,173	14·809
3	,,	Heaton Park†	,,	2·385	29,940	31,732	7,933	15·027
4	,,	Spinkwell	,,	2·329	Defective	Defective	—	15·388
5	,,	Idle Quarry‡	,,	2·464	42,484	43,380	10,845	14.545
6	,,	Jegrum's Lane‡	,,	2·400	45,172	47,860	11,965	14·933
7	,,	Spinkwell‡	,,	2·456	31,732	37,108	9,277	14·592
8	,,	Coppy Quarry ‡	,,	2·408	37,108	41,588	10,397	14·833
9	,,	Old Whatley †	,,	2·415	35,316	35,316	8,829	14·840
10	,,	Manningham Lane†	,,	2·401	28,148	37,108	9,277	14·927
11	,,	,,	,,	2·421	Failed	Failed	—	14·804
12	Grauwacke	Penmanmawr	,,	2·748	40,692	67,572	16,893	13·042
13	Granite	Mount Sorrel	,,	2·657	—	—	—	13·489
14	,,	,,	,,	2·675	51,444	51,444	12,861	13·398
15	Grauwacke	Ingleton	,,	2·787	35,316	(53,236)	Not crushed	12·866
16	Granite	Aberdeen.	,,	—	27,546	(28,340)	Not crushed	—
17	Syenite	Mount Sorrel	,,	—	47,284	47,284	11,821	—
18	Granite	Bonaw	1½	—	17,396	24,564	10,917	—
19	,,	Furness	,,	—	24,564	24,564	10,917	—
20	,,	A	,,	—	22,772	24,564	10,917	—
21	Limestone	B	,,	—	17,396	19,188	8,528	—
22	,,	C	,,	—	18,292	19,188	8,528	—
23	,,	Anston	1	—	2,154	3,050	3,050	—
24	,,	Worksop	,,	—	3,946	7,098	7,098	—
25	Sandstone	D	,,	—	3 050	3,498	3,498	—
26	,,	E	2	—	10,218	12,228	3,057	—

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* See a paper 'On the Comparative Value of various Kinds of Stone, as exhibited by their Powers of resisting Compression,' in the fourteenth volume of the 'Memoirs of the Literary and Philosophical Society of Manchester.'

† Pressure applied in the direction of the cleavage.

‡ Pressure applied perpendicularly on the bed of the stone.

On comparing the results on the Yorkshire sandstones, it will be seen that the difference of resistance to pressure does not arise so much from the variable character of the stone, as from the position in which it is placed as regards its laminated surface,—the difference being as 10 : 8 in favour of its being crushed upon its bed, to the same when crushed in the direction of cleavage; and the same holds true of the limestones.

A considerable number of the above specimens were dried and weighed; immersed in water, and again weighed, to ascertain their relative absorbing powers. The results are given in the following table:—

Experiments to ascertain the Amount of Water absorbed by different Kinds of Stone.

No. of Specimen.	Description of stone.	Locality.	Weight before immersion.	Weight after immersion for 48 hrs.	Difference of weight.	Proportion absorbed.
			lbs.	lbs.	lbs.	
1	Sandstone .	Shipley	5·4687	5·5546	·0859	1 in 63·6
2	”	Heaton	5·2578	5·3632	·1054	1 in 49·8
3	”	Heaton Park . .	5·1718	5·2896	·1171	1 in 44·1
4	”	Spinkwell . . .	5·2968	5·4726	·1758	1 in 30·1
5	”	Idle Quarry . .	5·7178	5·8203	·1016	1 in 56·3
6	”	Jegrum's Lane .	5·5976	5·7187	·1211	1 in 46·2
7	”	Spinkwell . . .	5·6757	5·7851	·1094	1 in 53·8
8	”	Coppy Quarry .	5·5703	5·6914	·1211	1 in 46·0
9	”	Old Whatley . .	5·4726	5·6132	·1406	1 in 38·9
10	”	Manningham Lane	5·4882	5·6093	·1211	1 in 46·3
11	”	”	5·6289	5·7539	·1250	1 in 45·0
12	Grauwacke .	Wales	5·4101	5·4140	·0039	1 in 1641·0
13	Granite . .	Mount Sorrel . .	5·6875	5·6992	·0117	1 in 485·0
14	”	”	5·8007	5·8124	·0117	1 in 495·0
15	Grauwacke .	Ingleton . . .	5·7500	5·7539	·0039	1 in 1962·6

It will be observed that Specimen No. 15, the Ingleton Grauwacke, is the least absorbent, and No. 12, the Welsh Grauwacke, absorbs almost as little, while Nos. 9 and 4 of the sandstones absorb most. The granites, though closely granulated, take up much more water than the Grauwacke, but less than the sandstones. The resistance of the Grauwacke specimens to the admission of water is four times that of the granite, and thirty-six times that of sandstone, such as is found in the Yorkshire quarries.

In this country we have perhaps some of the largest and most magnificent stone bridges in Europe, to prove which we have only to instance some of the constructions of one of the most successful bridge-builders of this or any other country, the late Mr. Rennie, also the numerous examples by Telford, Perronet, and other contemporaries. None, however, surpass, if any of them equal, the Waterloo and London bridges over the Thames; the first by the late, and the latter by the present Sir John Rennie. The arches of these bridges are elliptical, and whether they are considered in regard to their chaste and beautiful forms, their strength or their solidity of construction, they are undoubtedly among the finest and most perfect structures of their kind in the world. The flat arch has been greatly improved, and is much in demand in France and other parts of Europe, but for want of sufficient strength and solidity in the abutments, it has not always been successful. It requires the utmost strength and solidity in the abutments to oppose sufficient resistance to the thrust of the flat segmental arch. So far as beauty of design and stability of construction is concerned, we are much indebted to Perronet, the success of whose efforts is seen in the Pont St. Maixence, Pont de la Concorde, &c., both of which are fine and imposing structures. For elegance of design, however, they do not appear equal to those of Rennie, Telford, and other contemporaries.

Brick.

Brick, as a material for bridge building, may be considered as simply a substitute for stone, to be used in those districts where that material is not found, or is too expensive; as such, well-burnt bricks carefully moulded and made of good clay, are invaluable; and we might adduce numerous instances of their utility in securing despatch and economy in the erection of bridges. Well made and thoroughly burnt bricks appear to be almost as uninfluenced by hygrometric and thermometric changes as stone; they have this defect, however, that they cannot be used without the interposition of a large portion of plastic mortar between them, thus exposing the structure to considerable shrinking from the number of joints and the yielding nature of the material by which the parts are united. On account of this defect, brick as a bridge-building material will not bear comparison with stone; and their small parallelopiped form unfits them to some extent for the formation of arches, as they cannot, like stone, be at all times moulded to the form of voussoirs, radiating from the centre, and fitting one another accurately. Hence a bridge of bricks cannot be considered, theoretically or practically, equal to one which has only about a tenth of the number of joints, and in which the faces of the joints are parallel and perpendicular to the line of pressure.

The following experiments, a part of the series conducted by Mr. Latimer Clarke, already alluded to, gives the resisting power of this material:—

Brickwork.

	Lbs. per square inch.
1.—9 in. cube of cemented brickwork (Nowell & Co.), No. 1 (or best quality), weighing 54 lbs., set between deal boards. Crushed with 19 tons 18 cwt. 2 qrs. 22 lbs.	=551·3
2.—9 in. cube brickwork, No. 1, weighing 53 lbs., set in cement. Crushed with 22 tons 3 cwt. 0 qrs. 17 lbs.	=612·7
3.—9 in. cube brickwork, No. 3, weighing 52 lbs., set in cement. Crushed with 16 tons 8 cwt. 2 qrs. 8 lbs.	=454·3
4.—$9\frac{1}{4}$ in. brickwork, No. 4, weighing $55\frac{1}{2}$ lbs., set in cement. Crushed with 21 tons 14 cwt. 1 qr. 17 lbs. . . .	=568·5
5.—9 in. cube brickwork, No. 4, weighing $54\frac{1}{4}$ lbs., set between boards. Crushed with 15 tons 2 cwt. 0 qrs. 12 lbs.	=417·0
Mean	=521·0

Note.—The last three cubes of brick continued to support the weight, although cracked in all directions; they fell to pieces when the load was removed. All the brickwork began to show irregular cracks a considerable time before it gave way. The average weight supported by these bricks was 33·5 tons per square foot, equal to a column 583·69 feet high of such brickwork.

Wrought and Cast Iron.

Iron is a material which has only of late years been employed in the construction of bridges, except in the shape of straps and bolts for strengthening and binding together structures of wood. At the present time it has for many purposes nearly superseded all the others, and is almost exclusively employed. So extensive are its uses, and so generally is it employed in every department of art, that we may assert that no construction of any pretensions to strength and durability can be perfected without it.

It will not be necessary in this place to enter upon the strengths and other properties of iron, as the experimental inquiry has already been given in part in the former sections of this work, and the remainder will be more appropriately introduced when we arrive at the section which treats of wrought-iron bridges.

Cast iron has been employed in the construction of bridges for a longer period than wrought iron, as the bridges of Coalbrookdale (in 1773), Sunderland (in 1792-5), and Southwark (in 1815-9) testify. More recently, an immense number of cast-iron girder and arched bridges have been introduced for railways, of almost every possible form; and until late years they were the best means known to engineers for spanning wide rivers with a strong and rigid structure. Now the introduction of girders formed of wrought-iron plates, enables him to extend a level roadway across rivers and chasms of 400 or 500 feet wide, and we have probably yet to learn the limit to which these constructions may be carried by the judicious application of the wrought-iron system. It will not be necessary to enlarge upon the advantages to be derived from a more extended application of this material, which in strength and security, in lightness and economy, is so superior to cast iron, and every other material hitherto employed. Malleable iron, when properly applied, and in the most advantageous form, is three times as strong in its resistance to tension as the same weight of cast iron; and hence it appears that a third of the weight of the cast-iron structures might be saved by its employment, and in many cases a large portion of the expense. It is also more desirable than cast iron on account of its freedom from the flaws and cracks which are so frequent in all crystalline bodies when cast in large masses, which are subjected and exposed to unequal contraction in passing from the liquid to the solid state. Upon the danger from this cause we have already enlarged in the section on cast-iron beams.

It would be foreign to our purpose to trace the history of the art of bridge-building, or to endeavour to show the manner in which each of these primary materials, stone, brick, timber, and iron, has been employed in bridge construction. A very brief sketch of the application of iron to this purpose will be more consistent with the design of the present work and the limited space at our disposal.

The use of iron as the principal material in the construction of bridges seems to have suggested itself to Mr. Thomas Farnolls Pritchard, of Shrewsbury, as early as 1775, and the value of his conception was practically tested, soon afterwards, by the erection of the first cast-iron bridge over the Severn at Coalbrookdale. This undertaking, remarkable alike for its boldness of design, and magnitude of conception, proved perfectly satisfactory, and reflects the greatest credit upon its projectors. The bridge consists of five cast-iron ribs, each formed of a nearly semicircular arch of cast iron of 100½ feet span and 45 feet rise,—over the crown of which the railway is carried; —and of the parts of two other larger arches intercepted between the abutments and the roadway; the three arches are placed concentrically, and are connected together by radiating struts; they rest upon horizontal base plates, from which rise the vertical guides for keeping the arched ribs in position and communicating any lateral thrust to the abutments.

The large arch in the Coalbrookdale bridge was cast in two pieces, but, in 1790, Mr. Burdon designed a bridge for crossing the Wear at Sunderland, in which the radiating voussoirs of the stone arch were imitated by similarly formed perforated blocks of iron. The bridge consists of a single arch of 240 feet span, and only 30 feet rise, springing from 70 feet above low-water mark, and is composed of six cast-iron ribs placed parallel to each other at a distance of 6 feet apart. Each of these ribs consists of 105 blocks, formed as above described, and united together by wrought-iron arcs fitting in grooves and secured by bolts. The ribs are stayed together by diagonal bracing, the haunches being filled up by light cast-iron circles; over these a wooden framework supports the roadway. The structure is of extraordinary lightness, so much so, that after having stood the test of half a century, it is, we believe, under contemplation to strengthen it or replace it by a wider and more substantial structure. It contains 214 tons of cast and 46 tons of wrought iron, was completed in 1779, and is said to have cost 26,000*l*.

The success of these early attempts led to the very general adoption of iron for bridges, and the numerous examples in every part of the country attest the success of the application. For railway purposes, especially, and in cases where it is desirable to interfere as little as possible with the navigation below, the small versed sine or rise of the cast-iron arch offers great advantages. Hence it was that when it was determined to replace the old London Bridge, Messrs. Telford and Douglas sent in a design for a colossal cast-iron bridge of *six hundred feet span* and sixty-five feet rise; and in 1802, when the crossing of the Menai Straits first engaged the attention of engineers, Mr. Rennie sent in two plans for cast-iron bridges, one of a single arch of 450 feet span, the other of three arches of 350 feet span each. The expense (250,000*l*. to 290,000*l*.) caused the rejection of these last designs, and Telford's chain suspension bridge was erected in their stead.

There appears no reason why the cast-iron arch should not be employed in spans of such length, if necessary, although the largest cast-iron bridge with which we are acquainted is that of Southwark over the Thames. This noble structure consists of three spans, of 210, 240, 210 feet respectively, the centre arch rising 24 feet and the lesser ones 18 feet 10 inches. The roadway, 42 feet wide, is supported by 8 cast-iron ribs in each arch, connected by transverse and diagonal braces; the spandrils are filled up with diagonal framing on which rest cast-iron plates supporting the roadway. The bridge over the Neva at St. Petersburgh, although not approaching Southwark in extent of span, is in other respects of an even more colossal character. It consists of seven arches, varying in span from 107 to 156 feet; the width between the outside ribs is 66 feet, and the extreme length of the bridge between the abutments 1,078 feet.

It is estimated to contain, exclusive of the balustrades, &c., 6,928 tons of cast and 342 tons of wrought iron. This weight includes that of the Swivel Bridge on the right bank for the passage of ships.

The cast-iron arch cannot strictly be included in either of the classes into which we have divided bridges, inasmuch as, whilst from its arched form it partakes of the characteristic of the stone arch in deriving its strength from resistance to compression, yet it also approximates in a greater or less degree to the character of the beam, in which the lower side of the construction acts as a tie, opposing a tensile force to the thrust which in the stone arch is thrown exclusively upon the abutments. This gives to the cast-iron arch, which is in some degree homogeneous, the combined powers of resistance as a beam with those of a stone or brick arch, which are exclusively supported by the thrust upon the abutments.

For short spans up to 30 or 40 feet the arch-form is very generally abandoned, and horizontal girders employed to carry the roadway; this form of bridge, from its simplicity and level soffit, is peculiarly adapted for railway purposes, and hence attempts have been made to extend it to larger spans by introducing wrought-iron truss rods, either embedded in the lower flange of the girder itself, or, most frequently, attached to projections at the ends of the girder, as shown in the section on 'trussed girders.' Some consider these girders as safe in spans of 70 feet, but they are open to all the objections we have urged against similar beams in buildings, and can never be depended upon with certainty. The variations of temperature, or the passing of heavy loads, may at any time alter the relations existing between the truss and beam, and render them mutually inoperative. To all such combinations of different materials we have the most decided objections when a simple homogeneous material can be substituted.

Another method of employing cast iron in the construction of bridges, is that in which the roadway is suspended beneath a large cast-iron arch. Mr. Hosking considers this to be 'one of the least objectionable modes iron can be applied in construction with reference to its liability to expand and contract,' and 'one of the least expensive modes in which the metal can be used for wide spans and low headway underneath, whilst it gives a more firm and rigid roadway than any mode of suspension from chains has yet been made to yield.' These views may be considered as those current at a time when a more rigid structure was unknown; but in circumstances in which a ductile material like wrought iron can be united in such forms and combinations as to give the required rigidity, it becomes a question of considerable importance to what extent the cast-iron arch is applicable if the width of span and lightness of the edifice be a consideration. I apprehend that in such cases the simple tubular beam must have a decided preference over the cast-iron arch and suspended roadway.

Wrought iron appears scarcely to have come into use in the construction of cast-iron bridges, when its immense superiority in certain cases was so conclusively established by the long series of experiments which led to the erection of the magnificent structures which cross the Conway and Menai Straits. The adoption of Holyhead as the port of communication with Dublin, led to the commencement of the Chester and Holyhead Railway, in the construction of which an unexampled difficulty presented itself, viz., the crossing of the Menai Straits by a bridge of enormous span, and yet without either narrowing the channel or otherwise interfering with the navigation. A suspension bridge was too flexible for railway purposes, a cast-iron arch-bridge was rejected on account of the great difficulty of erection, and because the curvature of the arch at the haunches did not allow sufficient height of waterway, and it only remained to devise some entirely new construction with a clear horizontal soffit under the bridge. The way in which the difficulty was surmounted will be fresh in the recollection of our readers and need not be detailed here. Suffice it to observe that wrought iron was adopted as the *material*, and tubes through which the trains could pass as the *form* in which the bridge was to be constructed. A long series of experiments, part of which we have already given (Part II.), showed that the tubes should be made of rectangular form, and with a peculiar arrangement of the material at the top and bottom in long rectangular cells or tubes;—determined the amount of material required, and the formula for calculating the strength, and moreover proved that the tubes required no auxiliary support. Finally, the construction of the tubes having been completed, they were floated to their place and triumphantly raised to their present position across the Straits.

The principle of support in these bridges, it will be observed, is that of a simple beam or girder, supported by opposing a resistance to compression above, and to tension below, the neutral axis, and resting with a simple vertical pressure upon the abutments. The roadway is perfectly level, and as the distance from it to the under side or soffit of the bridge is only one or two feet, no other form of bridge so little interferes with the waterway. Its strength is uniform, certain, and easily determinable, and its span is practically speaking almost unlimited. These advantages have led to the very general adoption of tubular and tubular girder bridges, the only difference between them being that in the former the roadway or railway is carried through the tube itself, whilst in the latter the two or more tubes are placed parallel to one another, and the railway or roadway carried between them.

For small spans the tubular form is sometimes abandoned to secure simplicity of construction, and the simple plate girder similar to those employed in fireproof buildings, but of larger size, has been adopted. As we have already stated, we consider that in spans up to 150 feet, although the plate girders are inferior in strength to those of the tubular form, yet they are probably more advantageous, not only because they are of a more simple form, but because every exposed part can be reached for the purpose of painting—an operation essential to the durability of iron structures.

Another form called the Bowstring Bridge has met with favour in some quarters, and is somewhat similar to the arch suspension bridge we have already referred to. It consists of a hollow arch composed of wrought-iron plates riveted to angle irons, for resisting compression, and a similar hollow box below receiving the thrust of the arch and acting as a tie; the arch and horizontal tie are connected together by vertical and diagonal bracing, and two or three of these arches being placed parallel to each other, the railway or roadway is carried between them.

Recently, various modifications of the *lattice* form have been put forward as possessing great superiority over all others, and it may not be uninteresting to examine into the grounds upon which these assertions are made. We take for illustration the form known as Warren's girder, which consists of a line of cast-iron tubes on the top to resist compression, and a line of wrought-iron flat links or chains along the bottom to resist tension. The intermediate space or depth of the girder is filled up by a series of struts and ties of cast and wrought iron, placed alternately, so as to form with parts of the top and bottom a series of equilateral triangles, and connecting the two principal parts of the girder.

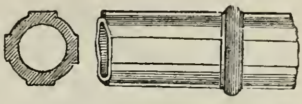
As much has been said and considerable pains taken to recommend this bridge as superior to others of different construction, it will not be inappropriate to examine into the advantages and disadvantages it presents, and apply the results to others of equal and some of greater merit. It has been urged that this description of bridge is above all others the most convenient for exportation, and probably with some reason, as it can be erected, marked, and taken to pieces at the manufactory, so as to obviate the necessity of sending out skilled workmen to put it together. And yet it may be questioned whether its numerous parts do not require more careful and accurate fitting than the simple riveting of the plates of a tubular bridge, and if it be shown to possess comparative weakness and other dangerous qualities, no consideration of convenience should induce the selection of an imperfect structure.

The chains, boring, and fitting the struts and ties and other objectionable parts of the manufacture must appear obvious when compared with a continuous, well riveted plate of iron, forming a simple homogeneous mass without movement in the joints and of uniform strength. If, for example, we take one of the largest and most effective bridges of this kind yet constructed, namely, that which carries the Great Northern Railway across Newark Dyke, a navigable branch of the Trent, we shall have an opportunity of comparing its resisting powers with that of a tubular girder bridge of similar weight and span, and

I make little doubt that the advantages will be seen to be in favour of the tubular girder bridge.

The Newark Dyke bridge is composed of four Warren's girders of 240 feet 6 inches span, 17 feet deep, and supported on eight cast-iron triangular frames resting on the piers or abutments on either side of the river. The top of each girder consists of a cast-iron tube, fig. 83, varying in diameter from 13½ inches at the ends to 18 inches at the centre. The bottom consists of a chain of wrought-iron links, 18 feet 6 inches in length from centre to centre of the holes, and 9 inches deep. The chain near the ends consists of 4 links 1 inch in thickness, and at the centre of 14 links ⅞ inch in thickness, the intermediate portions being graduated accordingly. At the ends the links are 16½ inches wide to receive the joint pin, 5½ inches in diameter. The chain is connected with the cast-iron tube by diagonal ties *a*, *a*, fig. 84, formed of links similarly to the chain, and similarly varied in strength. The end diagonal tie is composed of 4 links of an aggregate area of 32·6 inches, and the centre one of 2 links of 13·5 inches area. The diagonal struts *b*, *b*, alternating with the ties are of cast iron, cast in the form of a jaw at top to embrace the tube, and bored both at top and bottom for the reception of a joint pin 5½ inches in diameter. The girders are kept in position by horizontal bracing at top and bottom, consisting of a cast-iron tube connecting the opposite joints of each girder, and diagonal ties of wrought iron extending from each joint of either girder to the next in front of the other.

Fig. 83.



The weight of iron in the two girders forming one half the bridge is 138 tons 5 cwt. of cast, and 106 tons 5 cwt. of wrought; that is, the weight of iron in the four girders forming a bridge for two lines of railway is 489 tons: to this must be added the weight of platform, handrails, &c., 100 tons, making a total of 589 tons.

We have for calculation, therefore, the following dimensions:—

Length of span	240 feet 6 inches = 2,886 inches.
Depth of girder	17 feet = 204 inches.
Area of top in middle . .	142 square inches, cast iron.
Area of bottom in middle .	110 square inches wrought iron.

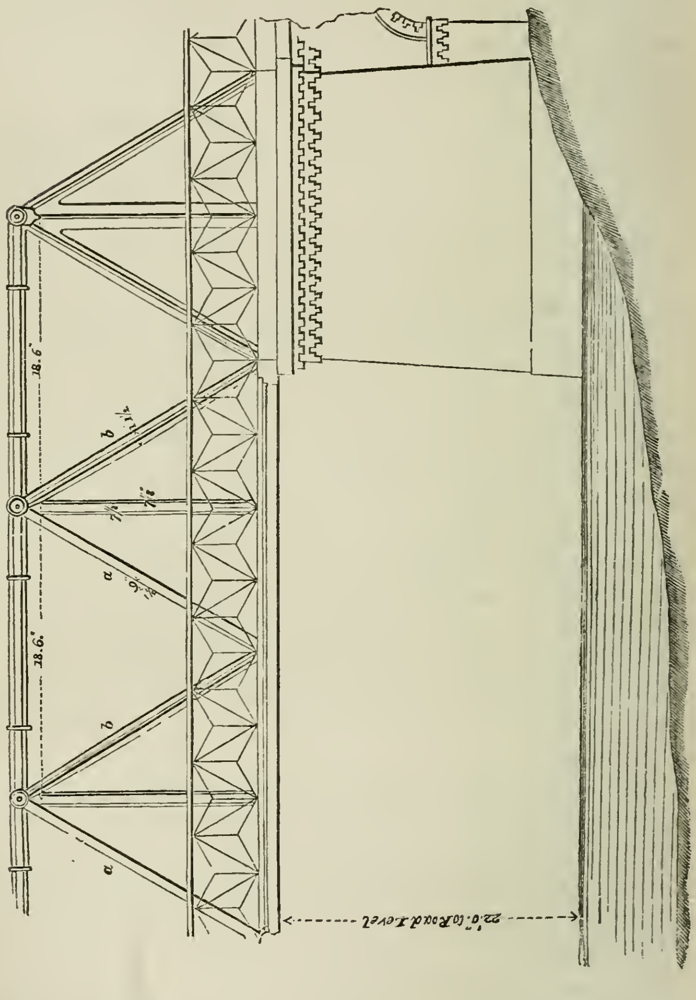


Fig. 84.

Now if we assume that the strengths of the top and bottom are properly proportioned, we may apply the formula for beams on the tubular principle, in order to obtain an approximation to the strength of the girder; by formula

$$\text{W}=\frac{a\,d\,c}{l}$$

$$=\frac{110\times 204\times 80}{2886}=622 \text{ tons,*}$$

the breaking weight of each girder at the centre. Now as there are four girders, the breaking weight of the bridge at the centre would be 2,488 tons, on the assumption that the diagonal struts and ties connected the top and bottom together as rigidly as the plates and T iron in a tubular bridge. From this should be deducted half the permanent load or weight of the bridge, leaving 2,194 as the actual weight, which placed at the centre would break the bridge. This result is too high for a bridge of this construction, and it would require a series of carefully conducted experiments to determine the exact law they follow.

Now a tubular girder bridge of 240 feet 6 inches span, sufficient to carry a double line of railway, and such as would require 2,880 tons equally distributed or 12 tons per foot run to break it, would be within the limits of 400 tons weight, or 189 less than the Newark Dyke Bridge; and although its strength is far less than that given by the formula for the latter, yet it is amply sufficient for the purpose.

There is, however, one point which would seem to indicate that the formula for tubular beams gives much too high a strength for the Warren girder, and that is the large deflection of this construction. With a load of 240 tons equally distributed over the bridge the deflection at the centre was $2\frac{3}{4}$ inches. Two heavy goods engines caused a deflection of $2\frac{1}{8}$ inches, and five of the heaviest class engines on the Great Northern produced a deflection of $2\frac{1}{2}$ inches. These experiments strikingly exhibit the weakness of this form of girder; in one on the tubular principle with solid sides the deflection upon the same span would not exceed $\frac{3}{4}$ of an inch. The large deflection which is observable in all girders with open sides, indicating a want of rigidity and union between the parts, is a very objectionable feature, and shows the great superiority of the tubular girder.

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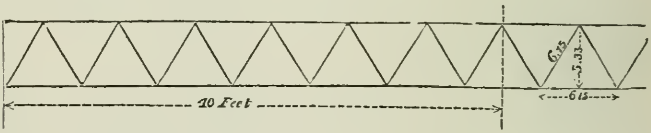
* Taking into account the greatly increased deflection of this description of girder as compared with that of the tubular girder, it is more than probable that 30 as a constant would more accurately represent the strength of the girder than 80 as given above. In this case W = 233 tons, only, a very different view of the advantage of this form of girder.

Irrespective of the weakness of the Warren girder, it is precarious from the fact that the failure of one strut or tie would endanger the whole structure, and hence, any accident, such as a train getting off the line, might bring down the entire bridge. Altogether we consider that there are dangerous elements of weakness in this bridge unless the number of struts and ties be increased, which would make it correspond with the lattice bridge. And if the number were increased still further we should arrive at a close approximation to the tubular girder with solid sides.

During the month of October, 1850, W. B. Blood, Esq., kindly sent me a mathematical analysis of the elements of strength in a Warren bridge, of which the following is an abstract.

'*Calculation of strain on Captain Warren's patent girder*, under load at centre of 250 tons, and uniform load of 500 tons. Span 80 feet.

Fig. 85.



'I assume from the tracing that the span of 80 feet is divided into 13 equal spaces, forming equilateral triangles. This gives each side of the triangle 6·15 feet, and the depth of the girder 5·33 feet.

Let $l=80$.
$d=5{\cdot}33$.
$\lambda=6{\cdot}15$.
$w=$Total load 250 or 500 tons, as the case may be.
$s=$Horizontal strain at centre.
$\Sigma=$Strain on a diagonal or lattice bar.

'First with load of 250 tons on centre, not taking into calculation the weight of the girder itself,

$$s=\frac{w\,l}{4\,d}=\frac{250\times 80}{4\times 5{\cdot}33}=938 \text{ tons.}$$

$$\Sigma=\frac{w\,\lambda}{2\,d}=\frac{250\times 6{\cdot}15}{2\times 5{\cdot}33}=144 \text{ tons.}$$

'This is uniform throughout the whole length, being alternately compression and tension on the alternate bars.

'Second with uniform load of 500 tons,

$$s=\frac{w\,l}{8\,d}=\frac{500\times 80}{8\times 5{\cdot}33}=938 \text{ tons.}$$

'The horizontal strain at any point is given by the formula $s=\frac{w}{2\,d\,l}(l\,r-n^2)$; r being distance of point from the abutment.

'The strain on the diagonals is expressed by the formula,

$$\Sigma=\frac{w}{d\,l}\,\lambda\,y$$

'If the uniform load is all laid along the top of the girder, y in the formula is the distance of the foot of the lattice bar from the centre of the girder. If the load is laid along the bottom, y is the distance of the top of lattice bar from the centre of the girder. If the load is half on the top and half on the bottom of the girder, y is the distance of the middle point of lattice bar from the centre of the girder.

'In this case we may consider the load as all on the top, which is the most favourable case for the girder.

'The bar marked a is under the greatest tension; for this bar $y=33{\cdot}82$ and

$$\Sigma=\frac{500\times 6{\cdot}15\times 33{\cdot}82}{80\times 5{\cdot}33}=244 \text{ tons.}$$

'If the weight were attached along the bottom, y would be 36·9, and

$$\Sigma=\frac{500\times 6{\cdot}15\times 36{\cdot}9}{80\times 5{\cdot}33}=266 \text{ tons.}$$

'I cannot make out the sectional area exactly, but I take the area of the bottom tie to be about 36 inches; and of the top and diagonals about 30 inches, or rather less.

'These would give with 500 tons uniform load on the top

32 tons per square inch compression of top,
26 „ „ tension of bottom,
8·2 „ „ tension of lattice bar *a*,

and with the load on the bottom nearly 9 tons per square inch, on lattice bar *a*.

'26 tons is too high for wrought iron, and according to Hodgkinson's experiments, the tensile strength of cast iron should not be assumed much over 6 tons per square inch, so that a uniform load on top of about 370 tons would break the bar *a*. My sectional areas however may not be correct. The use of cast iron as a tie is I think strongly to be condemned, and can only be considered safe where much superabundant strength is given, as the effect of vibration when under severe tension cannot be calculated. I apprehend the flanges at the points of connection with the top, will be apt to crack off with the working caused by deflection.

'It appears to me to be a most dangerous structure, and I hope the Government Inspector will not pass it.'

From the above it will be seen that Mr. Blood does not arrive at conclusions favourable to the Warren girder, although some of the defects he notices have been remedied in the larger example at Newark Dyke.

It will not be necessary to enter largely into the strength and other properties of the lattice bridge; we have already given it as our opinion that it is inferior in strength and power of resisting the violent vibratory action of a passing train, to the more solid structures of the plate and tubular girders. It is an improvement upon Captain Warren's more open structure, but it inherits, and that only in a less degree, the defects of that construction. In this bridge, as in Warren's, there is no protection for carriages and engines running off the line, as the lattice bars connected by one or more rivets are exceedingly weak and fragile; but in the tubular girder bridge with double sides, the chance of serious accident from this cause is small, as the whole bulk of the bridge is opposed to the shock of the engine.

Tubular and Tubular Girder Bridges.

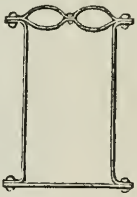
If the reader will turn back to Part II., on Wrought-Iron Beams, he will find detailed the principal part of the experiments which determined the form and proportions in which wrought iron should be combined so as most effectively to resist a transverse strain. These experiments conclusively showed:

1st. That the rectangular form of section was stronger than either the elliptical or circular.

2nd. That wrought-iron plates possessed very inferior powers of resistance to flexure in the direction of their thickness, and hence that, to secure uniformity of strength in a simple rectangular girder, the top is required to be about twice the sectional area of the bottom.

This weakness in the top of the tubes was the great difficulty which had to be surmounted in the experiments which led to the construction of tubular bridges. In these experiments the plates doubled up, or buckled, long before the entire compressive strength of the iron was called into operation. To obviate this difficulty, the experiments suggested the construction of cells or tubular corrugations extending longitudinally along the upper side of the tube. Such a tube having been made of the section shown in fig. 86, and having loaded it with increasing weights, it ultimately gave way by tearing the *sides* from the top and bottom at nearly the same instant of time after the last weight, 22,469 lbs., was laid on. The puckering of the top had been completely prevented, and greatly increased strength was the result.*

Fig. 86.



It is from this period we may date the disappearance of almost every difficulty respecting the construction of tubular and tubular girder bridges. The powerful resistance offered to compression by the cellular form of the top, as exhibited in this experiment, at once decided in my mind the form to be adopted for the large tubes which now span the Conway and the Menai Straits, and from this time forward I had no doubts as to the complete success of the undertaking.

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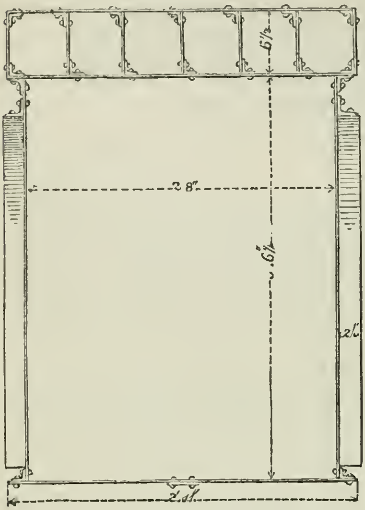
* This experiment is given in Part II., Experiment XXIX.

After this, the form of the Britannia bridge having been determined on, a tube was constructed on this principle, but in place of the corrugated top as exhibited in the last figure, rectangular cells were adopted, as shown in fig. 87, the intended section of the model tube one-sixth the size of one span of the Britannia bridge. The experiments on this tube bear directly on the strength of tubular bridges, and we therefore deem it necessary to give an abstract of the results.*

Fig. 87.



Dimensions.—Total length of tube, 78 feet. Breadth, 2 feet 11 inches. Depth, 4 feet 6 inches. Length between supports, 75 feet.

Thickness of bottom plates . . = 0·180 inch.
" side " . . = 0·099 "
" upper top " . . } = 0·147 "
" lower top " . . }
" division " . . }
Area of the top . . . . . . = 24·024 "
" bottom . . . . . = 8·800 "
" sides . . . . . . = 9·000 "
Weight of tube = 10,888·94 lbs.

Experiment I.—Broke after sustaining the load a minute and a half, the bottom plate tearing asunder 2 feet from the centre of the shackle:

Breaking weight, 79,578 lbs.
Ultimate deflection, 4·5 inches.

In this experiment, it will be observed, the area of the top was three times that of the bottom, and, as might have been expected, it gave way at the latter part. It was immediately repaired, and the bottom strengthened by two additional plates each $6\frac{1}{2} \times \frac{5}{16}$ inches in section, so that the area of the bottom was now 12·8 inches.

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* For a detailed account, see my work on the Britannia and Conway Bridges pp. 251 *et seq.*, and Plates XV. to XX.

Experiment II.—Gave way at one end by twisting out of the perpendicular.

Breaking weight, 97,102 lbs.

In this experiment, the tube failed from want of stiffness in the sides. It was therefore repaired and strengthened by the introduction of vertical bars of $1\frac{1}{2}$ inch angle-iron riveted to the sides internally at every 2 feet. A Saint Andrew's Cross was also inserted at each end.

Experiment III.—Broke, the plates tearing asunder a few inches from the bottom, with 126,128 lbs.

Ultimate deflection, 5·79 inches.

The tube was again repaired and two stronger slips riveted along the bottom, 20 feet on each side of the centre, in order to enlarge the sectional area of the middle and to force the top to yield to compression; these strips were 9 inches wide and half an inch thick, which increased the area of the bottom to 17·8 inches.

Experiment IV.—Broke with 148,129 lbs., the bottom tearing through the solid plates. The sides were also damaged. On examination the 9-inch strips were found to be defective, having more the appearance of cast than wrought iron.

Ultimate deflection, 4·94 inches.

The fractured parts were cut out and replaced by stronger and better plates, and as the top cells and plates were considerably damaged, they were straightened, and in some cases the injured parts renewed. An entirely new bottom extending 20 feet on each side of the shackle was introduced, composed of double plates one-fourth of an inch thick, with two strips along the middle as before; by these the area was increased to 22·45 inches in the middle.

Experiment V.—Not broken with 129,007 lbs.

This experiment was discontinued in order to ascertain the lateral strength of the tube, and set at rest the question as to the effect of violent winds upon the bridge. It was accordingly laid upon its side and the weights suspended by a shackle as before.

Experiment VI.—A weight of 26,781 lbs. was laid on and left for some hours, during which the deflection increased from 2·36 to 2·5 inches. On relieving the tube of its load, the permanent set was found to be only one-tenth of an inch.

Having attained satisfactory results as to the lateral strength of the tube, it was restored to its vertical position and again loaded.

Experiment VII.—A load of 135,255 lbs. was left suspended upon the tube for nine days and nights in order to ascertain the effects of long-continued strain. During this period the deflection increased from 3·17 to 3·22 inches.

The loading was then continued and the tube gave way with 154,452 lbs. by tearing asunder through the end of new plates, 21 feet 6 inches from the shackle. Area of bottom at point of fracture 8·8 inches.

Ultimate deflection, 3·86 inches.

In this experiment the tube gave way from a want of due proportion in the material of the bottom. The tube was repaired and additional plates were added, extending a few feet nearer the supports on each side of the centre of the tube.

Experiment VIII.—Broke with 192,892 lbs. = 86¼ tons, the cellular top puckering at a distance of 2 feet from the shackle, the bottom and sides remaining uninjured.

Ultimate deflection, 4·89 inches.

This experiment determined the relative proportions of the top and bottom areas of the tube so as to balance the forces of extension and compression developed by a transverse strain, and furnished other data necessary for the construction of a tube having a maximum strength with a given quantity of material. Hitherto the top had always had considerably more strength than the bottom; now both were of about uniform strength, the ratio of the areas of the top and bottom being as 24 : 22 or 12 : 11.

Now if we calculate the value of C, in this experiment, and compare it with the former experiments, we shall find that

For the cylindrical tubes, it = 13·03 tons.
For the elliptical tubes . = 15·3 „
For the rectangular tubes . = 21·5 „
For the model tube . . = 24·4 „

This increase of strength, as indicated in the experiments on the model tube, was evidently due to the cellular construction of the top. In the rectangular tubes the difficulty had been the small resistance offered to flexure by thin plates in the direction of their thickness; this was overcome by placing the plates in the direction in which they offered a maximum resistance to flexure, viz., horizontally, to prevent lateral buckling by compression, vertically, to prevent similar injury in that direction; and again, by the use of angle-irons securely riveted to the plates, the whole of that part was made perfectly rigid and secure from flexure and from the tendency to '*pucker*,' which was always present in the former experiments with the solid top.

To determine the best *form* for the cells, Mr. Hodgkinson made some elaborate and carefully conducted experiments, some of which are here inserted, as still further illustrating the necessity of the cellular formation in any construction where the tops of wrought-iron girders are subjected to severe compressive strain.

Table I.—Experiments made to determine the resistance of plates (or bars) of wrought iron to a force of compression; the plates being placed in a vertical position, with their ends made perfectly flat, so as to be well bedded against two parallel and horizontal crushing surfaces.

Vertical length of plate.		Lateral dimensions of plate.	Area of plate.	Breaking weight.	Weight per square inch of section borne by plate at time of fracture.
ft.	in.	inches.	sq. in.	lbs.	tons.
10	0	2·98 × ·503	—	1,222	·364
10	0	3·01 × ·766	2·306	7,793	1·508
10	0	2·99 × ·995	2·975	12,735	1·911
10	0	3·00 × 1·51	4·53	46,050	4·538
5	0	2·98 × ·507	1·511	8,469	2·502
0	7½	1·023 × 1·023	1·0465	50,946	21·733

The above experiments, it will be seen, give very remarkable results: the crushing strength of the same plates varies from 0·364 to 21·733 tons per square inch, according as the breadth, length, and thickness of the plates are altered. Mr. Hodgkinson derives from them the following laws:—

1st. The breaking weight is nearly inversely proportional to the square of the length.

2nd. The breaking weight varies as the cube of the thickness.

3rd. The breaking weight varies directly as the width.

In the following table 'the lateral dimensions of the cells are so large that, with a length of 10 feet, the pillars were not destroyed by flexure, but by absolute buckling or crushing.'

Table II.—Resistance of Rectangular Tubes, all 10 feet long, to a Force of Compression in the Direction of their Length.

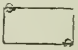
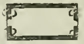
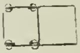
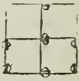
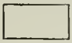
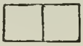
External dimensions of tube.	Thickness of plates of tube.	Weight with which buckling or perceptible undulation was observed.	Weight of greatest resistance.	Form of section of tube.	Area of section of tube.	Weight per square inch of greatest resistance.
inches.	inch.	lbs.	lbs.		inches.	tons.
4·1 × 4·1 4·1 × 4·1 4·25 × 4·25 4·25 × 4·25	·03 ·06 ·083 ·134	— — 29,290 46,314	5,534 19,646 37,354 51,690		·504 1·0200 1·484 2·3947	4·902 8·5986 11·237 9·636
8·175 × 4·1 8·5 × 4·75	·061 ·264	13,209 —	23,289 197,163		1·532 7·326	6·786 12 015
8·4 × 4·25	·26 & ·126	99,916?	206,571 =92·2 tons		6·89 nearly	13·3845
8·1 × 4·1 $8\frac{1}{2} \times 4\frac{3}{4}$	·059 $\frac{1}{4}$ nearly	37,401 —	43,673 —		1·885 8·3466	9·877 (Not crushed with 11·12 tons)
8·1 × 8·1 8·37 × 8·37 8·5 × 8·375 8·5 × 8·4	·06 nearly ·139 ·2191 ·245 & ·238	15,897 82,475 — —	27,545 100,395 198,955 —		2·070 4·9262 7·7367 8·4665	5·926 9·098 11·48 (Not crushed with 11·015 tons)
8·1 × 8·1 " "	·0637 "	56,630 46,635	70,070 82,027		3·551 "	8·809 10·312

TABLE III.—RESISTANCE OF CIRCULAR TUBES, ALL 10 FEET LONG, TO A FORCE OF COMPRESSION IN THE DIRECTION OF THEIR LENGTH.

No. of tube.	External diameter of the tube.	Internal diameter of the tube.	Thickness of the plates of the tube.	Weight of greatest resistance.	Area of section of tube.	Weight of greatest resistance per square inch.
	inches.	inches.	inches.	lbs.	inches.	tons.
1	1·495	1·292	—	6,514	·4443	6·55
2	1·964	1·755	—	14,158	·6104	10·35
3	2·49	2·275	—	23,958	·8045	13·29
4	2·35	1·865	—	34,516	1·605	9·60
5	2·34	1·91	·215	31,828	1·4353	9·901
6	2·995	2·693	—	37,356	1·349	12·362
7	4·05	3·772	—	47,212	1·7078	12·34
8	4·06	3·75	·150 nearly	49,900	1·9015	11·71
9	6·366	6·106	·1298	91,402	2·547	16·021
10	6·187	—	·0939	60,075	1·799	14·908

From the above it will be seen that it is essential to have the thickness of the plates duly proportioned to the size of the cells; and that doubling the thickness of the plates (other things being the same) is far from giving double the strength per square inch of the section. It is also evident that the strength of the square and rectangular tubes decreases as the size of the cells is increased; that the rectangular form  is the weakest, the strength of which being nearly doubled by the addition of a division across the centre, thus ; and that the circular form is the strongest of all. The rupture of the small cylindrical tubes was, doubtless, partly due to the flexure which they underwent, so that small and large cylinders cannot be expected to follow the same law of resistance to a force of compression.

On this question Mr. Tate, however, has shown that, as forming the top of a tubular bridge, the square form is theoretically stronger than the circular, and comparing this result with that of Mr. Hodgkinson's experiments he makes the following observations:—'The cells in a transverse strain undergo a different kind of strain to what they are subjected to in a *simple crushing* force, equally distributed over the section of the tube. In this case, all the parts of the section are equally compressed, and it is reasonable to conclude that the best form of the cell will be that in which the material is equally distant from the axis of pressure; but the case of transverse strain is very different; the upper edge undergoes the greatest strain, and, of the other parts, that which is nearest the neutral axis of the beam undergoes the least: in this case, therefore, the material in the square cells is symmetrically distributed with respect to the axis of pressures—the neutral axis of the beam.' * It is further to be observed, that in the cells of tubular bridges an approximation is made to the circular form by the introduction of angle-irons at the corners, which, without altering the symmetrical arrangement of the material with respect to the neutral axis, nevertheless strengthen the square cells at their weakest parts.

The following experiment upon a cell of the same dimensions as those in the Conway and Britannia bridges will be of interest. The tube was 18 inches square, 8 feet long, and composed of plates half-an-inch thick, riveted to angle-irons at each corner. It was crushed by an hydraulic press of the best construction, the use of which was kindly offered by Messrs. Benjamin Hick and Co. of Bolton.

Sectional area of the cell = 50 inches.

The following table exhibits the pressure and flexure of the tube under the influence of the force applied:—

No. of Experiment.	Pressure in tons.	Flexure on side A.	Flexure on side B.	Compression from force applied in inches.	Remarks.
1	115	—	—	—	The discrepancies exhibited on the sides were in a great measure occasioned by the 'buckling' of the plates, which produced contortions of the surface.
2	165	·25	·20	·000	
3	215	·25	·48	·000	
4	265	·25	·70	·000	
5	315	·25	·70	·025	
6	365	·25	1·20	·032	
7	415	·25	·95	·032	
8	465	·25	·70	·030	
9	515	·50	·95	·042	
10	565	·75	·95	·061	A short time before the whole amount of pressure came upon the tube it was progressively yielding to the strain. It ultimately became distorted by the puckering of three of the sides.
11	615	·75	1·75	·063	
12	665	1·00	—	·160	
13	690	—	—	—	

Taking 680 tons as the crushing force, we have the resistance of compression per square inch $= \frac{680}{50} = 13·6$ tons.

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* Strength of Materials, p. 76.

From these facts it would appear that the cellular system, when applied to the upper side of wrought-iron beams subjected to transverse strain, is the only true principle on which to obtain the greatest power of resistance to compression with the least quantity of material. And this is the more strikingly apparent from the experiments on various kinds of hollow beams, which elicited the principles on which the tubular system is founded. On comparing such natural objects as combine *strength* with *lightness*, they will be found to embody this principle in a greater or lesser degree; the hollow bones of birds, for instance, and the stems of grasses, grain, and bamboos are so formed, and in order to fulfil the designs of the great Architect of Nature, follow the same unerring laws as have been illustrated in the construction of the Conway and Britannia bridges. Indeed all bones when examined microscopically are found to be composed of minute cells, adding greatly to the strength and lightness of the structure, and exhibiting a conformation of wonderful adaptation to the work they have to perform.

As an illustration we have only to refer to the annexed sketch, figs. 88, 89, of how Nature works in supplying defects, whether natural or accidental, in giving the required stability to all her structures where strength is required. This evidently is the case from the appearance of the two longitudinal sections of thigh bones, taken from a rickety subject, where distortion had taken place.

In this case it will be observed that, in order to compensate for the bent form or curvature of the bone (which in a more healthy state, to act as a pillar, would have been nearly straight), the whole of the porous or cellular interior is incased in a thin shell or tube of hard bony substance, as dense and compact as the finest ivory. Had the subject been healthy and the limb straight, the envelope of ivory would have been thin, and something like the form shown at *a a*; but owing to the curvature, and in order to compensate for that defective form in its resistance to vertical pressure, Nature, in her workings, supplies the deficiency by filling up the concave side with a thicker stratum of hard bone, and a proportionate thinner stratum on the convex side, to make up for the loss of strength arising from the curvature of the pillar, as shown at *b b*. It is thus that the great Mechanician of Nature works, and it is thus that we should follow the dictates of an unerring power, from which we cannot deviate without incurring the risk of failure in all the modifications of constructive art.*

Fig. 88.

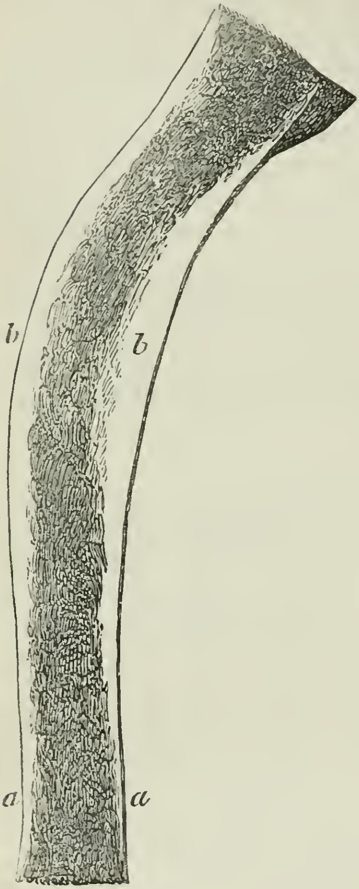
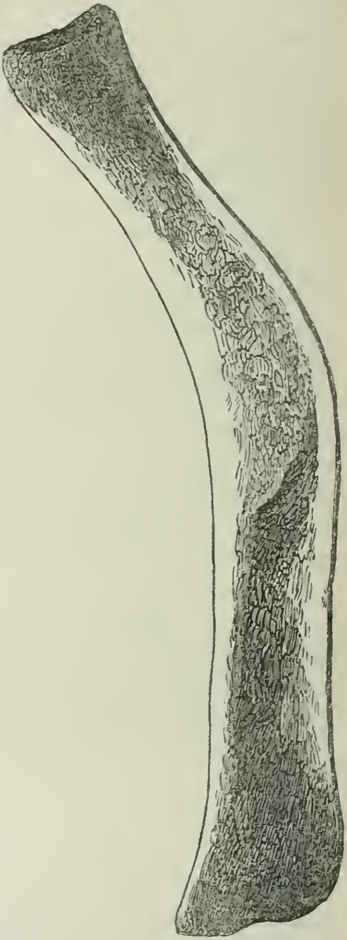


Fig. 89.



On the subject of wrought-iron bridges it is to be observed, that notwithstanding the advantages peculiar to the use of cells in the construction of beams or bridges of large dimensions, in practice it is found necessary to confine that form to bridges of wide span or where the width exceeds 100 or 150 feet; within these limits the same objects are most economically obtained by the use of thicker plates and an increase of weight. The cells in this case are abandoned, not in order to obtain strength, but to obviate difficulties in the construction, and other objections to their use, such as their small size, and the impossibility of cleaning, painting, &c., to prevent oxidation. Under other circumstances, and where the span exceeds 100 to 150 feet, the cellular construction is highly important; and indeed, with large spans, becomes indispensable for securing lightness, combined with strength and economy in the use of the material.

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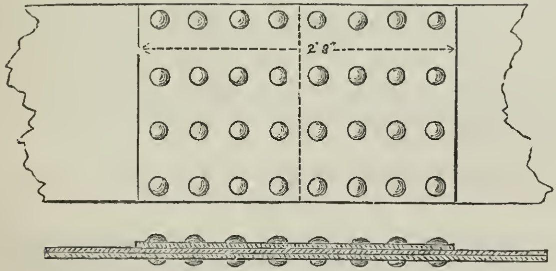
* See Mr. Fairbairn's description of the Tubular Crane.—*Transactions of the British Association for the Advancement of Science*, Sections in Report, 1850, p. 177.

The Bottom.—In the formation of the bottom of a tubular girder, whether composed of cells, as in the Britannia and Conway bridges, or of double plates, as in smaller examples, it is of importance to have as few joints as possible. Hence the plates should be rolled as long as their weight and thickness will allow, and the joints be carefully united by covering plates, *chain-riveted,* as shown in the annexed sketch, fig. 90, with three or more rows of rivets, according to the width of the plates. Eight rivets are required in each of the lines, four on each side of the joint, to give sufficient strength, and the area of the rivets collectively should be equal to the area of the jointed plates, taken transversely through one line of the rivets, the area of the parts punched out in that line being deducted. These

Fig. 90.



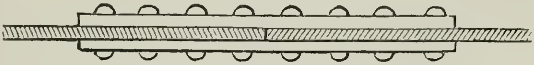
proportions give the required security to the joint, and afford nearly the same strength to a tensile strain as the solid plate; that is, if the covering plates be as much thicker as will give the same area of section through the rivet holes as the imperforated double plate. These precautions being taken in covering the joints of the plates and in securing the angle-irons which unite the sides with the bottom, it will meet in practice all the requirements of a uniform power of resistance to strain from one end of the girder to the other.

In a long experimental enquiry which I undertook some years since, it was shown that there was a loss in the riveted joint, as compared with the solid plate, of 30 to 50 per cent.; that is, taking the strength of the solid plate at 100, that of the double-riveted joint would be 70, and that of the single-riveted joint 50.

This great deficiency in the strength of joints subjected to a transverse strain, caused considerable difficulty in designing the Britannia and Conway bridges; double, treble, and quadruple riveting was thought of; but one after another was abandoned, on account of the rivet holes weakening the plates; and I should almost have despaired of attaining the object in view, but for the system of longitudinal or chain-riveting, having occurred to me, after repeated trials of other modes and forms. Experiment, however, established the perfect security of this method, as the following tables clearly demonstrate. Two distinct methods were tried, one with a single thickness of plates, the joint having a covering strip on each side; the other with two thicknesses of plate, there being a joint in one of them covered by a plate, and kept in position by a line of rivets, as already described. The jointed plates having been prepared, the experiments were effected by a powerful lever, tearing the joints and plates asunder in the direction of the line of rivets.

Chain-riveting. Single Plates, with double Covers over the Joint.

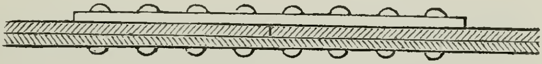
Area of section through solid plate $3{\cdot}5 \times {\cdot}25 = {\cdot}875$ sq. in.
Area of the covering plates . . $3{\cdot}5 \times {\cdot}26 = {\cdot}910$ „
Area of section through rivet hole $3{\cdot}0 \times {\cdot}25 = {\cdot}750$ „
Diameter of the rivets, each $\frac{1}{2}$ inch, four on each side of the joint.

No. of Experiment.	Weight in lbs.	Elongation in inches.	Remarks.
1	5,600	—	Weight of the lever.
2	26,656	—	
3	28,448	—	
4	30,240	·021	
5	32,032	·034	
6	33,824	·034	
7	35,626	·044	
8	37,418	·052	
9	39,210	·056	
10	41,002	—	Torn asunder through a rivet hole after sustaining the load a few seconds.

If we take the area of the plate at the point of fracture= ·750 inch, it will be found that it required a power of 24·41 or nearly 24½ tons per square inch to tear it asunder.

Chain-Riveting. Double Plates and a single Cover over the Joint.

Area of section through plates 2 × ·875 = 1·750 sq. in.
Area of section through rivet holes . = 1·5 ,,
Area of covering plate through rivet hole = 0·91 ,, = 0·785
Rivets as before, ½-inch diameter.

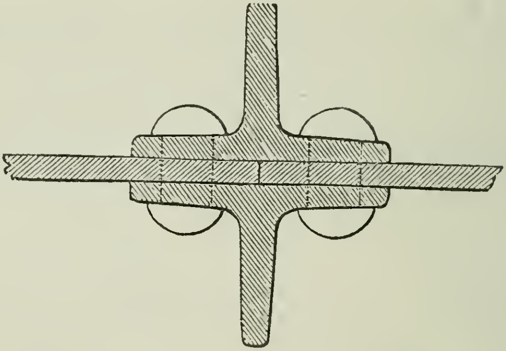
No. of Experiment.	Weight in lbs.	Elongation in inches.	Remarks.
1	5,600	—	Weight of lever.
2	26,656	·016	
3	37,408	·025	
4	46,368	·028	
5	55,328	·075	
6	62,496	·100	
7	69,664	—	Broke by shearing off the rivets close to the plate.

From the above experiment, it appears that fracture took place through the solid plate on one side, and by shearing off the rivets on the other. Hence the area of section of fracture= ·875 × ·785 = 1·66 inches, and, proceeding as before, we have 18·73 tons per square inch as the breaking weight.

Finding the resisting powers of the rivets unequal to the strength of the double plates, they were afterwards increased from half an inch to five-eighths of an inch in diameter, or until the area of the rivets approached nearly to the area of the plates, which gave the required strength. In joints of this description it will be found that the resisting powers of the rivets is nearly equal to that of the plates, i.e. the resisting power of the rivet is to that of the plates as their sectional areas respectively. This is in agreement with the following laws, which have been deduced from experiment: (1st) that the ultimate resistance to shearing, in any bolt or rivet, is proportional to the sectional area of the bar torn asunder; and (2nd) that the ultimate resistance of any bar to a shearing strain is nearly the same as the ultimate resistance of the same bar to a direct longitudinal tensile strain.

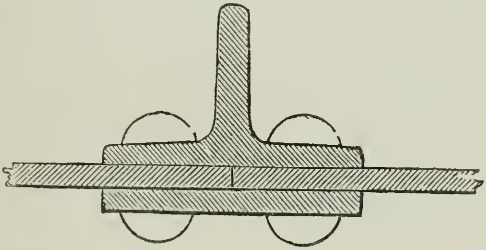
The Sides.—It has been argued by some that the sides of a tubular girder, or the rib of a cast-iron beam, have no other office to perform than to retain the top and bottom of the girder in position, and that they add nothing to its strength. Hence, also, they have contended for the use of open lattice-work, as sufficiently rigid to maintain the required connection between the top and bottom flanges. Now, I think it can be shown that the sides of either plate or tubular girders have a much more important office to perform: that they not only maintain the top and bottom flanges in their relative positions, but they contribute largely to the strength of the beam, and from them mainly arises the vertical rigidity of the structure. The sides of a tubular girder, when securely riveted to double T iron, as in bridges of large span, fig. 91, or with a T iron inside and

Fig. 91.



a covering strip outside, as in bridges of smaller span, fig. 92, are not only calculated to retain the top and bottom in their places but by the intervention of vertical T irons at distances of every two or three feet, a rigid prop or pillar is introduced to maintain the form and to distribute the strain over the top and bottom of the girder. The whole of the sides, therefore, above and below the neutral axis, assist, in the ratio of their sections and of their distance from the neutral axis, to resist tension on the one hand, and compression on the other. These results are deduced from experiment; but taking the lattice, or Warren, or any other girder with open sides, it will be found that the whole of the strain is upon the top and bottom, whilst the sides are more or less inadequate for the purpose of maintaining the proper distance between the two extreme lines of tension and compression, and consequently there is a greatly increased deflection from the same weight. The lattice bridge was a very imperfect structure when first introduced into this country, and it has only attained to its present condition since the true principles of construction have been developed by the experiments on the great tubular bridges of North Wales. Even now it is questionable whether it is equal in its powers of resistance to the tubular girder or the solid plate beam. It is decidedly inferior in stiffness or resistance to flexure, and, from the observed increase of its deflection under strain, it is evidently weaker than the solid girder with plate sides.

Fig 92.



Every structure having for its object public convenience and the support of a public thoroughfare, should possess within itself the elements of an undeniable security. Bridges and viaducts should especially contain those elements, as they are peculiarly liable to accident, and from whatever cause such accident may arise, the community must be equally interested in the strength and durability of the structure. In the introduction of a new system of construction, comprising the use of new and comparatively untried materials, it behoves the projector, on public grounds, to be careful and attentive to the most minute circumstances directly or indirectly affecting the security of the bridge. In those of the tubular construction, considerations of this kind are of primary importance, as much depends, not only upon the principle of construction, but upon the quality of the material employed, and of the workmanship introduced, which in every case should be of the very best description.

In the construction of tubular bridges I have endeavoured to apply these principles; and having a strong conviction of their great superiority in strength, durability, and cheapness, I have not hesitated to advocate their employment and extended application under every circumstance where great strength and lightness is required. It however becomes necessary from time to time to submit them to a rigid examination, and, before opening such bridges as public thoroughfares, it is essential to subject them to severe and satisfactory tests. These tests and examinations have been various and frequent, and it may safely be affirmed that in no case, where tubular bridges have been duly proportioned and well executed, has there been the least reason to doubt their security.

It has already been determined by experiment, that in order to balance the two resisting forces of tension and compression in a wrought-iron tubulur girder, having a cellular top, that the sectional area of the bottom should be, to the sectional area of the top, as 11 : 12; which being the correct relative proportion, it then follows that any increase to the one without a proportionate addition to the other, will involve a useless waste of material, inasmuch as increased weight is given to the girder by the introduction of material which in this instance is almost entirely unproductive. This being the case, it is of importance to preserve, as nearly as possible, the correct relative proportion of the parts, in order to ensure the maximum of strength in the two resisting forces of tension and compression.

This appears to be an arrangement essentially important in these structures, and also in the application of the formula to determine the ultimate strength of the bridge. If, for example, an excess was given to the bottom of a girder, the formula

$$\mathrm{W} = \frac{a\,d\,c}{l}$$

would not apply, as the top and bottom areas would be disproportionate to each other, and that in excess would have to be reduced to the due proportion of 11 : 12; or, in other words, the additional strength must be omitted from the calculation in computing the strength of the bridge. The same reasoning is in operation where the excess of area happens to be in the cellular top, although in this case the formula $\mathrm{W} = \frac{a\,d\,c}{l}$ applies, with equal force as the excess must be omitted as before in the calculation of the strength of the girder.

Assuming, however, that these proportions are maintained, the above formula furnishes a correct principle on which to estimate the strength of wrought-iron tubes of this description, whatever may be their depths or relative dimensions.[*]

In the case of Torksey tubular bridge, for instance, which was objected to on account of its instability by the Government Inspector, there is a want of proportion in the areas of the top and bottom, as the following numbers show:—

Sectional Area of the Top.

	Ft.	In.	In.	Sq. In.
Longitudinal plates . .	2	$8\frac{5}{8} \times 2 \times$	$\frac{3}{8}$	$=24{\cdot}47$
Vertical plates . . .	1	$1\frac{1}{4} \times 3 \times$	$\frac{5}{16}$	$=12{\cdot}42$
Angle iron . . .	0	$4\frac{3}{4} \times 9 \times$	$\frac{5}{16}$	$=13{\cdot}35$
Area of cellular top, as given by Mr. Fowler .				50·24
Ditto Capt. Simmons				51·72
Mean . . .				50·98

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[*] Mr. Tate remarks upon this formula .—

1st. With respect to $\mathrm{W} = \frac{a\,d\,c}{l}$, where a is the section of the bottom, $c = 80$, the constant deduced on this supposition, will apply to all depths of the tube, within short limits of error, where such depths (or d) are large in proportion to the depth of the cells and the thickness of the plates.

Sectional Area of the Bottom.

	Ft.	In.	In.	Sq. In.
Longitudinal plates . . .	2	9 ×	$\frac{5}{8}$ × 2 =	41·25
Centre strip	1	0 ×	$\frac{3}{4}$ =	9·00
Packing strip	0	$3\frac{3}{4}$ ×	$\frac{10}{16}$ × 2 =	4·68
				54·93

Span . . . . . . 130 feet.

Here there is an evident want of proportion, the bottom being greatly in excess of the top, which renders a reduction of the area of the bottom of the girder from 54·93 to 46·76 absolutely necessary. Hence by the formula,

$$\text{W} = \frac{a\,d\,c}{l}, \text{ or } \frac{46{\cdot}76 \times 120 \times 80}{1560} = 287{\cdot}7 \text{ tons},$$

or 288 tons = the breaking weight in the middle of one girder. From this is obtained 288 × 4 = 1152 tons as the breaking weight equally distributed over one of the spans of Torksey bridge, neglecting the weight of girders, ballast, rails, chairs, &c. ; which are differently estimated, but must be deducted from the breaking weight of the bridge.

Mr. Fowler estimates an equal distribution of the load on one span of the Torksey bridge as follows :—

	Tons.	Tons.
Rails and chairs	8	= 177
Timber platform	15	
Transverse beams	27	
Ballast, four inches thick	35	
Half the weight of the four girders, which are each 46 tons in weight (it should have been the whole weight when equally distributed)	92	
To this must be added the rolling load as agreed upon		= 195
Total . .		372

2nd. With respect to the formula $\text{W} = \frac{\text{A}\,d\,\text{C}}{l}$, where A is the area of the whole cross section, and C = 26·7, then the tubes should be similar in all respects, but a slight variation in depth, from that of a similar form, will not produce much error, especially where the depth is considerable. At the same time it must be observed, that both formulæ apply with great exactness where the tubes are similar.

Now as the ultimate strength of the bridge is 1,152 tons, it follows, that 177 tons, the permanent load, will reduce its bearing powers to 1,152 — 177 = 975 tons, as a resisting force to the heaviest rolling load that can be brought upon the bridge, being in the ratio of 975 : 195 or 5 : 1. These appear to be the facts of the case; and although the principal girders do not attain the standard of strength which I have ventured to recommend as the limit of strength, they are nevertheless sufficiently strong to render the bridge perfectly secure. In the calculations for estimating the strength of bridges of this description, it is always assumed that the proportions of the top and bottom of the girder are not only correct, but that the sides are sufficiently rigid to retain the girder in shape. It is further assumed that the whole of the plates are in the line of the forces, and that the workmanship and riveting are good.

On the excess of strength that should be given to girder bridges, there is a difference of opinion. I, however, entertain the conviction, that no girder bridge should be considered safe, unless it be calculated to sustain six times the greatest load that can be brought upon it; and in wrought-iron tubular girder bridges the breaking weight should be calculated at 12 tons to the lineal foot, inclusive of the weight of the bridge, or about *six times* the maximum load.

The following tables exhibit the strengths, proportions, and other properties of the girders, which are recommended in structures of this kind, and for spans from 30 to 300 feet.

The first column gives the length of the clear span from pier to pier.

The second, the breaking weight of the bridge in the middle.

The third, the area of the plates and angle iron of the bottom of the girder.

The fourth, the area of the cellular top; and

The last column the depth of the girder in the middle.

TABLE SHOWING THE PROPORTIONS OF TUBULAR GIRDER BRIDGES, FROM 30 TO 150 FEET SPAN; WHERE THE DEPTH OF THE GIRDER IS $\frac{1}{13}$TH THE SPAN.*

Span.	Centre breaking weight of bridge.	Sectional area of bottom of one girder.	Sectional area of top of one girder.	Depth at the middle.	
Feet	Tons	Inches	Inches	Ft.	In.
30	180	14·63	17·06	2	4
35	210	17·06	19·91	2	8
40	240	19 50	22·75	3	1
45	270	21·94	25·59	3	6
50	300	24·38	28·44	3	10
55	330	26·81	31·28	4	3
60	360	29·25	34·13	4	7
65	390	31·69	36·97	5	0
70	420	34·13	39·81	5	5
75	450	36·56	42·67	5	9
80	480	39·00	45·50	6	2
85	510	41·44	48·34	6	7
90	540	43·88	51·19	6	11
95	570	46·31	54·03	7	4
100	600	48·75	56·88	7	8
110	660	53·63	62·56	8	6
120	720	58·50	68·25	9	3
130	780	63·38	73·94	10	0
140	840	68·25	79·63	10	9
150	900	73·13	85·31	11	6

TABLE SHOWING THE PROPORTIONS OF TUBULAR GIRDER BRIDGES, FROM 160 TO 300 FEET SPAN; WHERE THE DEPTH OF THE GIRDER IS $\frac{1}{15}$TH THE SPAN.

Span.	Centre breaking weight of bridge.	Sectional area of bottom of one girder.	Sectional area of top of one girder.	Depth at the middle.	
Feet.	Tons.	Inches.	Inches.	Ft.	In.
160	960	90·00	105·00	10	8
170	1,020	95·63	111·56	11	4
180	1,080	101·25	118·13	12	0
190	1,140	106·88	124·69	12	8
200	1,200	112·50	131·25	13	4
210	1,260	118·13	137·81	14	0
220	1,320	123·75	144·38	14	8
230	1,380	129·38	150·94	15	4
240	1,440	135·00	157·50	16	0
250	1,500	140·63	164·06	16	8
260	1,560	146·25	170·63	17	4
270	1,620	151·88	177·19	18	0
280	1,680	157·50	183·75	18	8
290	1,740	163·13	190·31	19	4
300	1,800	168·75	196·88	20	0

* I have generally taken the depth of the girders at $\frac{1}{15}$th of the span; but in cases where the span does not exceed 150 feet it has been found more economical to adopt $\frac{1}{13}$th of the span. For spans above 150 feet it is, however, more conve-

In these tables the breaking weights of all the girders are calculated by the formula $W = \frac{a\,d\,c}{l}$; as, for example, taking from the table a bridge similar to Torksey, 130 feet span, a, = the area of the bottom = 63·38 inches; d = depth of the girder = 120 inches; c = 80 the constant deduced from experiment, and l = length of clear span = 1,560 inches. Then for each girder the breaking weight in the centre or $W = \frac{63{\cdot}38 \times 130 \times 80}{1560} = 390$ tons. Equivalent to 780 tons on each girder, or 1,560 tons, when the load is equally distributed over the surface of the platform of the bridge. If from this we deduct 190 tons as the permanent load of the bridge, there remains a strength of 1,370 tons as a resisting force to the travelling load of 195 tons; or in other words, the strength of the bridge is rather more than seven times the greatest weight that can be passed over it.*

Another subject of importance is the force of impact and the effect of vibration on bridges of this description. I am of opinion that the principles upon which I have endeavoured to establish the construction of these particular bridges, ever since their first introduction, adequately provide against any danger from this cause, and may be relied upon as sufficient to meet all the requirements of railway traffic. In several carefully conducted experiments on tubular girder bridges, of spans varying from 60 to 100 feet, the deflection was found to be as nearly as possible the same at all velocities. On this subject an elaborate series of experiments was made by the Commissioners on

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nient, on account of the great weight of the girder, to adhere to the original proposition of $\frac{1}{15}$th, in order to keep the centre of gravity as low as possible, and to prevent oscillation under a passing load. In situations where it is objectionable to increase the depth of the girder, it then becomes essential to increase the sectional areas of the bottom and top, in the ratio of the depths.

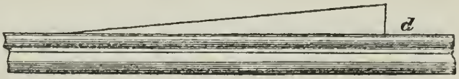
* Since the above table was completed, 1 ton per lineal foot has been taken as the permanent weight of bridges from 40 feet up to 100 feet span, and the rolling load as 2 tons per lineal foot; and in spans varying from 100 to 300 feet, the permanent weight of the bridge is estimated at 1½ tons per lineal foot, and the rolling load also at 1½ tons per lineal foot. For practical purposes these proportions are found to be perfectly safe; although in spans above 300 feet where the permanent weight of the structure becomes a large proportional of the load, it becomes necessary to introduce into the calculation new elements as regards strength, as may be seen in those for the Britannia and Conway tubular bridges.

railway structures, which showed an enormous increase of the deflection as the velocity increased to 30 miles an hour. It is however to be observed that these experiments were on small bars 9 feet long and 4 inches broad, and although they are highly valuable and exceedingly interesting, I am of opinion that there must be considerable difference in the effects of a weight rolling over a cast-iron bar and over a bridge 60 or 100 feet long. It is true the Commissioners, in their report, have qualified the results obtained from these experiments, by others upon existing cast-iron girder bridges where the deflection was reduced from an increase of the statical deflection, amounting to $\frac{9}{10}$ths of an inch, produced on the 9 feet bars at a velocity of 30 miles an hour to ·03 of an inch, upon a bridge of 48 feet span, at a velocity of 50 miles an hour; thus clearly showing, that the larger the bridge and the greater the rigidity and inertia of the girders, the greater will be the reduction of the deflection due to a passing load. In the tubular girder bridges the Commissioners had no experience, nor were they acquainted with the strength, rigidity, and other properties of girders, composed of wrought-iron riveted plates. In these bridges, so far as my experiments go, the deflection due to the passing load appears to be the same at all velocities, and unless there exists irregularities and inequalities on the rails, tending to cause a series of impacts, it may reasonably be concluded, that the deflections are not seriously, if at all, increased at high velocities.

On the effects of impact I concur in opinion with the Commissioners, that the deflections produced by the striking body on wrought iron are nearly as the velocity of impact, and those on cast iron greater in proportion to the velocity. These experiments and investigations are extremely valuable.

In confirmation of what has here been stated I may add the results of some experiments on one of the earliest of these tubular girder bridges, that over the turnpike road at Blackburn. The bridge consists of three girders, and is 60 feet 6 inches in span. Three locomotive engines, weighing 60 tons, were coupled together, so as to occupy the entire span, and passed over the bridge at velocities varying from 5 to 25 miles an hour. This load produced a deflection of ·3 of an inch, and that without any sensible increase in the deflection arising from the different rates of speed. Two wedges or inclined planes were then fitted to the rails, fig. 93, at the middle of the bridge, and the engines run over them at rates of from 1 to 10 miles an hour. The shock of the engines as they respectively fell upon the girders from a height of 1 inch, the thickness of the wedge at *d*, gave an increase of deflection from ·3 to ·42 inch; and another set of wedges $1\frac{1}{2}$ inches in height gave a further increase of deflection from ·42 to ·54 of an inch.

Fig. 93.



The mode of testing the bridges is a part of the inquiry which requires consideration, and, in order to maintain unimpaired the elastic powers of the structures, the tests should not exceed the greatest load the bridge is intended to bear at high velocities. In fact, the Commissioners are correct in assuming that the flexure of the girders should never exceed one-third of their ultimate deflection. In wrought-iron girders the effects of reiterated flexure are considerably less, in a well-constructed bridge of similar proportions to those given in the above tables, than those of cast iron. The deflection produced in these constructions by the greatest load will not be more than one-sixth of the ultimate flexure of the girder.*

It will be desirable that our readers should be acquainted with the mathematical analysis upon which the formula $W = \frac{a\,d\,c}{l}$ rests, that they may be able to judge for themselves of its value, and of the truth of the charge that it is merely *empirical.* We therefore give the investigation of the formula by Mr. Tate, undertaken by him during the construction of the Britannia and Conway tubular bridges.

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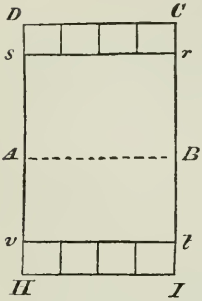
* Since the above was written a long series of experiments has been made to determine the endurance of wrought-iron girders subjected to vibration and alternate changes of load. These experiments clearly show that time is an element in the endurance of every description of bridge or girder composed of iron; or any other material where the strains extend to a disturbance of its molecular construction.—See *Philosophical Transactions* for 1864.

Investigation of Formulæ relative to Rectangular Tubes.

Let H I C D represent a section of the tube with a cellular structure; A B the neutral axis;



W = the breaking load of the tube;

l = the distance between the supports;

e = D s, the depth of the top cells;

e_1 = H v, the depth of the bottom cells;

p = the whole breadth of the spaces in the top cells;

k = the thickness of the plates;

a = the area of the section of the material in the top cells D C r s;

a_1 = the area of the section of the material in the bottom cells H I t v: and so on to similar notation for the parts below the neutral axis;

f = the force per square inch opposed to compression at the centre of the top cells;

S = the force per square inch opposed to compression at the upper edge D C;

g = the distance of the centre of the top cells from the neutral axis A B;

h = A D, the distance of the top of the tube from the neutral axis;

G = the distance between the centres of the top and bottom cells;

M = the moment of resistance of the section H I C D to rupture.

Hence we have, neglecting the material in the sides v s and t r, resistance of material in D C r s to compression = resistance whole area D C r s − resistance space in the cells.

$$=\frac{bf}{g}\int_{g-\frac{e}{2}}^{g+\frac{e}{2}}\{x\,dx\}-\frac{pf}{g}\int_{g-\frac{e}{2}+k}^{g+\frac{e}{2}-k}\{x\,dx\}$$

$$=\frac{bf}{g}\left\{\left(g+\frac{e}{2}\right)^2-\left(g-\frac{e}{2}\right)^2\right\}-\frac{pf}{2g}\left\{\left(g+\frac{e}{2}-k\right)^2-\left(g-\frac{e}{2}+k\right)^2\right\}$$

$$=\frac{bf}{2g}\,.\,2\,g\,e-\frac{pf}{2\,g}\,.\,2\,g\,(e-2\,k)$$

$$=f\,\{b\,e-p\,(e-2\,k)\}=f\,a.$$

Similarly, we have,

Resistance of material in H I $t\,v$ to extension $=f_1\,a_1$

$$\therefore\ f\,a=f_1\,a_1 \quad . \quad . \quad . \quad . \quad (1.)$$

Moment resistance of D C $r\,s$ to compression, m,

$$=\frac{f\,b}{g}\int_{g-\frac{e}{2}}^{g+\frac{e}{2}}\{x^2\,d\,x\}-\frac{f\,p}{g}\int_{g-\frac{e}{2}+k}^{g+\frac{e}{2}-k}\{x^2\,d\,x\}$$

$$=\frac{f\,b}{3\,g}\left\{\left(g+\frac{e}{2}\right)^3-\left(g-\frac{e}{2}\right)^3\right\}-\frac{f\,p}{3\,g}\left\{\left(g+\frac{e}{2}-k\right)^3-\left(g-\frac{e}{2}+k\right)^3\right\}$$

$$=f\,b\,e\,g\left\{1+\frac{e^2}{12\,g^2}\right\}-f\,p\,(e-2\,k)\,g\left\{1+\frac{(e-2\,k)^2}{12\,g^2}\right\}.$$

Without infringing upon the peculiarity of the structure we may suppose that k is indefinitely small, and that the material is chiefly collected in the vertical plates connecting D C and $s\,r$.

Then we have

$$m=\{f\,b\,e\,g-f\,p\,(e-2\,k)\,g\}\ \left(1+\frac{e^2}{12\,g^2}\right)$$

$$=f\,g\,\{b\,e-p\,(e-2\,k\},\text{ neglecting }\frac{e^2}{12\,g^2}{}^*$$

$$=f\,a\,g.$$

Similarly, we have,

Moment resistance of H I $t\,v$ to extension $=f_1\,a_1\,g_1$,

$$\therefore\ \text{M}=f\,a\,g+f_1\,a_1\,g_1\,;$$

but by equation (1.)

$$f\,a=f_1\,a_1,$$

$$\therefore\ \text{M}=f\,a\,(g+g_1)$$

$$=f\,a\,\text{G or }f_1\,a_1\,\text{G}\,; \quad . \quad . \quad . \quad (2.)$$

$$\therefore\ \frac{\text{W}\,l}{4}=f\,a\,\text{G},$$

---

* In the model tube Experiment XLI.,

$$e=6{\cdot}5,\ g>27\ -\ 3{\cdot}2>23{\cdot}8,$$

$$\therefore\ \frac{e^2}{12g^2}>\frac{6{\cdot}5^2}{12\times 23{\cdot}8^2}\angle\frac{1}{160}.$$

Hence it appears that the portion of the above formula which is rejected, is less than $\frac{1}{160}$th part of that which is retained, so that the relation $m=f\,a\,g$ is sufficiently exact for all practical purposes.

$$\therefore\ \mathrm{W}=\frac{4\,f\,a\,\mathrm{G}}{l},\ \text{or}\ \frac{4\,f_1\,a_1\,\mathrm{G}}{l}\quad .\ .\ .\quad (3.)$$

Substituting $\mathrm{S}\,\frac{g}{h}$ for f, we find,

$$\mathrm{W}=\frac{4\,\mathrm{S}\,a\,d}{l}\cdot\frac{g\,\mathrm{G}}{h\,d};$$

but in all practical cases $\frac{g\,\mathrm{G}}{h\,d}$ equals unity very nearly;

$$\therefore\ \mathrm{W}=\frac{4\,\mathrm{S}\,a\,d}{l}=\frac{a\,d\,\mathrm{C}}{l}.$$

Again, in similar tubes f must be constant, for $f=\mathrm{S}\times\frac{g}{h}=$ $\mathrm{S}\times$ a constant; and moreover G must be some constant proportional part of the depth of the tube; hence $4\,f\,\mathrm{G}=d\times$ a constant $=d\,\mathrm{C}$.

$$\therefore\ \mathrm{W}=\frac{a\,d\,\mathrm{C}}{l},\quad .\ .\ .\ .\ .\ .\ .\quad (4.)$$

where C must be determined by experiment.

Moreover, as the area of the whole section of the material in similar tubes must be some constant proportional part of a, we also have

$$\mathrm{W}=\frac{\mathrm{A}\,d\,\mathrm{C}}{l},\quad .\ .\ .\ .\ .\ .\ .\quad (5.)$$

whence we have for the value of the constant

$$\mathrm{C}=\frac{\mathrm{W}\,l}{\mathrm{A}\,d},\quad .\ .\ .\ .\ .\ .\ .\quad (6.)$$

The general formula (5.) will afterwards be shown to hold true for cylindrical and elliptical tubes.

In the model tube, Experiment VIII., page 216, the neutral axis must obviously be very nearly in the centre of the section. It will therefore be interesting to investigate a formula for this case. For this purpose, let $\mathrm{I}=$ the moment of inertia of the section H I C D, and $k_2=$ the sum of the thicknesses of the side plates $v\,s$ and $t\,r$; then we have, by a well-known formula,

$$\frac{\mathrm{W}\,l}{4}=\frac{\mathrm{S}\,\mathrm{I}}{h}\quad .\ .\ .\ .\ .\ .\ .\quad (7.)$$

$$\therefore\ \mathrm{W}=\frac{8\,\mathrm{S}\,\mathrm{I}}{l\,d}\quad .\ .\ .\ .\ .\ .\quad (8.)$$

But the moment of inertia of the section $=2\,\{$moment A B C D $-$ moment space A B $r\,s-$ moment space in the cells D C $r\,s\}$;

$$\therefore \mathrm{I}=2\left[b\int_{o}^{h}\{x^2\,dx\}-(b-k_2)\int_{o}^{h-e}\{x^2\,dx\right.$$
$$\left.-p\int_{h-e+k}^{h-k}\{x^2\,dx\}\right]$$

$$=\frac{2}{3}\left[b\,h^3-(b-k_2)(h-e)^3-p\,\{(h-k)^3-(h-e+k)^3\}\right]. \quad . \quad (9.)$$

which expresses the value of I in equation (8.).

The value of the constant S deduced from (7.) is

$$\mathrm{S}=\frac{\mathrm{W}\,l\,h}{4\,\mathrm{I}}$$
$$=\frac{\mathrm{W}\,l\,d}{8\,\mathrm{I}} \quad . \quad . \quad . \quad . \quad . \quad (10.)$$

As an application of equation (6.) and equation (10.) let us take the data of Experiment VIII.

Ex. In Experiment VIII., $\mathrm{W}=89{\cdot}15$ tons, $\mathrm{A}=55{\cdot}47$, $d=4{\cdot}5$ ft., $l=75$ ft., $b=35$ in., $k={\cdot}147$ in., $k_2={\cdot}189$ in., $e=6\frac{1}{2}$ in.

Whence we have from equation (6.)

$$\mathrm{C}=\frac{\mathrm{W}\,l}{\mathrm{A}\,d}=\frac{89{\cdot}15\times 75}{55{\cdot}47\times 4{\cdot}5}=26{\cdot}7 \text{ tons.}$$

And from equation (10.), we have

$$b-k_2=35-{\cdot}198=34{\cdot}802,\ h=\frac{4{\cdot}5\times 12}{2}=27,$$

$$h-e=27-6{\cdot}5=20{\cdot}5,\ p=35-7\times{\cdot}147=33{\cdot}971,$$
$$h-k=27-{\cdot}147=26{\cdot}853,\ h-e+k=20{\cdot}5+{\cdot}147=20{\cdot}647,$$

$$\therefore \mathrm{I}=\frac{2}{3}\left[35\times 27^3-34{\cdot}802\times 20{\cdot}5^3-33{\cdot}971\,\{26{\cdot}853^3-20{\cdot}647^3\}\right]$$
$$=20200 \text{ nearly;}$$

$$\therefore\ \mathrm{S}=\frac{\mathrm{W}\,l\,d}{8\,\mathrm{I}}=\frac{89{\cdot}15\times 75\times 12\times 4{\cdot}5\times 12}{8\times 20200}=26{\cdot}8 \text{ tons.}$$

Here it will be seen there is a very near coincidence between the values of C and S.

When the tube is simply a hollow rectangular beam, similar to that of Experiment XIV., p. 87, we find from equations (8.) and (9.) by making $e=2\,k$

$$\mathrm{I}=\frac{2}{3}\,\{b\,h^3-(b-k_2)\,(h-2\,k)^3\}\,;$$

substituting $\frac{d}{2}$ for h, putting b_1 for $b-k_2$ = the internal breadth, and d_1 for $d-4\,k$ = the internal depth, we have

$$\text{I}=\frac{1}{12}\{b\,d^3-b_1\,d_1^{\,3}\} \quad . \quad . \quad . \quad . \quad . \quad (11.)$$

$$\therefore \text{W}=\frac{8\,\text{S}\,\text{I}}{l\,d}$$

$$=\frac{2\,\text{S}}{3\,l\,d}\{b\,d^3-b_1\,d_1^{\,3}\}$$

$$=\frac{2\,\text{S}}{3\,l\,d}\{b\,d^3-(b-2\,k)\,(d-2\,k)^3\}$$

$$\therefore \text{S}=\frac{3\,\text{W}\,l\,d}{2\,\{b\,d^3-(b-2\,k)\,(d-2\,k)^3\}} \quad . \quad (12.)$$

by substituting $b-2\,k$ for b_1, and $d-2\,k$ for d_1.

Ex. In an experiment, $l=30\times 12$, $\text{W}=\frac{1{\cdot}212}{2}+22{\cdot}75=23{\cdot}356$ tons, half the weight of the tube + the breaking weight; $d=24$, $b=16$, and $k={\cdot}272$, hence we have by equation (12.),

$$\text{S}=\frac{3\times 23{\cdot}356\times 30\times 12\times 24}{2\{16\times 24^3-(16-{\cdot}544)\,(24-{\cdot}544)^3\}}=14 \text{ tons nearly.}$$

Formulæ relative to Cylindrical Tubes.

As the thickness of the metal in these tubes is uniform, we shall suppose that the neutral axis passes through the centre of the circular section.

Let r, r_1 = the radii of the exterior and interior circles respectively;
d, d_1 = the diameters of the exterior and interior circles respectively;
k = the thickness of the metal,
A = the area of the section of the material,
x, y = the co-ordinates of a point in the circle referred to the centre as the origin.

The other notation being the same as in the preceding investigation; then we have

$$\text{M}=\frac{2\,\text{S}}{r}\int_{-r}^{r}\{y\,x^2\,d\,x\}-\frac{2\,\text{S}}{r}\int_{-r_1}^{r}\{y\,x^2\,d\,x\}$$

$$=\frac{2\,\text{S}}{r}\cdot\frac{\pi\,r^4}{8}-\frac{2\,\text{S}}{r}\cdot\frac{\pi\,r_1^{\,4}}{8}$$

$$=\frac{\text{S}\,\pi}{4\,r}\left\{r^4-r_1{}^4\right\}$$

$$=\frac{\text{S}\,\pi}{4\,r}(r^2-r_1{}^2)(r^2+r_1{}^2)$$

$$=\frac{r\,\text{S}}{4}(\pi\,r^2-\pi\,r_1{}^2)\left\{1+\left(\frac{r_1}{r}\right)^2\right\}.$$

$$=\frac{r\,\text{S}\,\text{A}}{4}\left\{1+\left(\frac{r_1}{r}\right)^2\right\}.$$

Now in similar tubes $\frac{r_1}{r}$ is a constant quantity, and consequently $1+\left(\frac{r_1}{r}\right)^2$ is also a constant quantity; in this case therefore we have

$$\text{M}=\frac{\text{A}\,d\,\text{C}}{4}$$

$$\therefore\ \frac{\text{W}}{4}=\frac{\text{A}\,d\,\text{C}}{4}$$

$$\therefore\ \text{W}=\frac{\text{A}\,d\,\text{C}}{l}\quad\cdots\cdots\quad(13)$$

which is the same general formula as that given in equation (5.)

When the thickness of the tube is small as compared with its depth, then $1+\left(\frac{r_1}{r}\right)^2=2$ very nearly,[*] and in this case,

$$\therefore\ \text{M}=\frac{r\,\text{S}\,\text{A}}{4}\left\{1+\left(\frac{r_1}{r}\right)^2\right\}=\frac{r\,\text{S}\,\text{A}}{2},$$

$$\therefore\ \text{W}=\frac{\text{A}\,d\,\text{S}}{l}$$

Comparing this expression with (13) we find $\text{C}=\text{S}$.

Formulæ relative to Elliptical Tubes.

Let a = Semi-axis major of the exterior ellipse.
b = Semi-axis minor of the exterior ellipse.

---

[*] In Experiment IV., on Conway and Menai Tubular bridges, p. 217, $d = 18{\cdot}26$ $k = {\cdot}0582$;

$$\therefore r=\frac{18{\cdot}26}{2}=9{\cdot}13,\ r_1=9{\cdot}13-{\cdot}0582=9{\cdot}072,$$

and $1+\left(\frac{r_1}{r}\right)^2=1+\left(\frac{9{\cdot}072}{9{\cdot}13}\right)^2=1{\cdot}99$ or 2 nearly.

a_1 = Semi-axis major of the interior ellipse.
b_1 = Semi-axis minor of the interior ellipse.
d = The depth of the tube.
k = The thickness of the metal.
A = The area of the section of the material, &c.

Proceeding precisely as in the case of the cylindrical tubes, we have

$$\text{M} = \frac{\text{S}\,\pi}{4\,a}\{b\,a^3 - b_1\,a_1{}^3\}\,;$$

now we shall assume that the exterior and interior ellipses in the transverse section of the tube are similar; hence in this case $\frac{a}{b} = \frac{a_1}{b_1}$;

$$\therefore \text{M} = \frac{\text{S}\,\pi}{4\,b}\{b^2\,a^2 - b_1{}^2 a_1{}^2\}$$

$$= \frac{\text{S}\,\pi}{4\,b}(b\,a - b_1\,a_1)\,(b\,a + b_1\,a_1)$$

$$= \frac{a\,\text{S}\,\pi}{4}(b\,a - b_1\,a_1)\left(1 + \frac{b_1\,a_1}{b\,a}\right)$$

$$= \frac{a\,\text{S}\,\text{A}}{4}\left\{1 + \frac{b_1\,a_1}{b\,a}\right\}.$$

Now in similar tubes $\frac{b_1\,a_1}{b\,a}$ is a constant quantity,

$$\therefore \text{M} = \frac{\text{A}\,d\,\text{C}}{4},$$

$$\therefore \frac{\text{W}\,l}{4} = \frac{\text{A}\,d\,\text{C}}{4}$$

$$\text{W} = \frac{\text{A}\,d\,\text{C}}{l}, \quad . \quad . \quad . \quad . \quad . \quad (14),$$

which is the same general formula as that given in equations (5) and (13).

When the thickness of the tube is small as compared with its depth, then $1 + \frac{b_1\,a_1}{b\,a} = 2$ very nearly,[*] and in this case

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[*] In Experiment XIX., on Conway and Menai Tubular bridges, p. 224, $a = \frac{14{\cdot}62}{2} = 7{\cdot}31$, $b = \frac{9{\cdot}25}{2} = 4{\cdot}625$, $k = {\cdot}0412$, $a_1 = 7{\cdot}31 - {\cdot}0416 = 7{\cdot}2684$, $b_1 = 4{\cdot}625 - {\cdot}0416 = 4{\cdot}5834$,

and $1 + \frac{b_1 a_1}{b\,a} = 1 + \frac{4{\cdot}5834 \times 7{\cdot}2684}{4{\cdot}625 \times 7{\cdot}31} = 1{\cdot}98$ or 2 nearly.

$$\mathrm{M} = \frac{a\,\mathrm{S}\,\mathrm{A}}{4}\left\{1 + \frac{b_1\,a_1}{b\,a}\right\} = \frac{a\,\mathrm{S}\,\mathrm{A}}{2},$$

$$\therefore\ \mathrm{W} = \frac{\mathrm{A}\,d\,\mathrm{S}}{l}$$

Comparing this expression with (14), we find $\mathrm{C} = \mathrm{S}$.

From equations (5), (13), and (14), it appears that

$$\mathrm{W} = \frac{\mathrm{A}\,d\,\mathrm{C}}{l}$$

is a general expression for the breaking weight of all tubes, whether rectangular, cylindrical, or elliptical, under the limits specified; where A is the area of the section of the material in square inches, d = the depth in linear inches, l = the distance between the points of support in linear inches, and C a constant determined by experiment for the particular form of the tube.

Hence the value of the constant C in those expressions, may be taken as the index of the comparative strengths of the different kinds of tubes.

To express the breaking weight of a Tube, as compared with the weight of the Tube itself.

Let s = the weight of a cubic foot of wrought iron, and w = the weight of the tube; then, supposing the tube to be uniform in its dimensions, we have

$$\mathrm{W} = \frac{\mathrm{A}\,d\,\mathrm{C}}{l} = 144\ \mathrm{A}\,l\,s \cdot \frac{d}{l^2} \cdot \frac{\mathrm{C}}{144\ s} \cdot = w \cdot \frac{d}{l^2} \cdot \frac{\mathrm{C}}{144\ s},$$

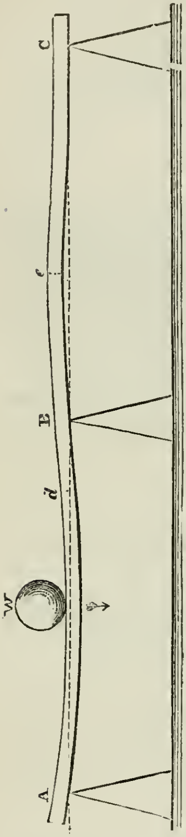
$$\therefore \frac{\mathrm{W}}{w} = \frac{d}{l^2} \cdot \frac{\mathrm{C}}{144\ s} = \frac{d\,\mathrm{C}_1}{l^2},$$

where the ratio varies as the depth of the tube, and inversely as the square of the length.

Continuity of Girders over two or more Spans.

It will be observed that, in calculating the strengths of a tubular or tubular girder bridges, the effect of making the girder continuous over more than one span has been purposely omitted. This view of the matter does not, however, prevent our investigation of the value of the counterpoise in its resistance to the load, in cases where the bridge has more than one span. The importance of the auxiliary strength thus obtained is acknowledged by all authorities, and the mathematician will find it an element which in his calculations could not with any propriety be neglected.

Fig. 94.



If, for example, we suppose a beam extending over two spans A B and B C continuously and having to support a weight W at the centre of one span, it then follows, that the top of the beam would be compressed between A *d* and *e* C, but that in the intermediate space *d e* between the points of contrary flexure, the top would have to support a tensile strain, caused by the weight W. In the position shown there would be a tendency (supposing the material to be sufficiently elastic) to raise the whole or part of the girder extending over the span B C into an arched form, and the weight of this portion would in turn act as a counterpoise to the weight W. On this subject Mr. Pole laid before the Institute of Civil Engineers a mathematical investigation of considerable interest, which I have quoted, in order to place before our readers the means of estimating for themselves the importance of the continuous principle in the erection of tubular bridges.

*Investigation of general Formulæ applicable to the Torksey Bridge.**

A beam of uniform section, and of perfectly elastic material, is supported horizontally at three points, A B and C (fig. 95), the support B being midway between the others. The two spans A B and B C are each loaded with different weights, distributed uniformly over the length of each span respectively, the weight on the part A B being the greatest.

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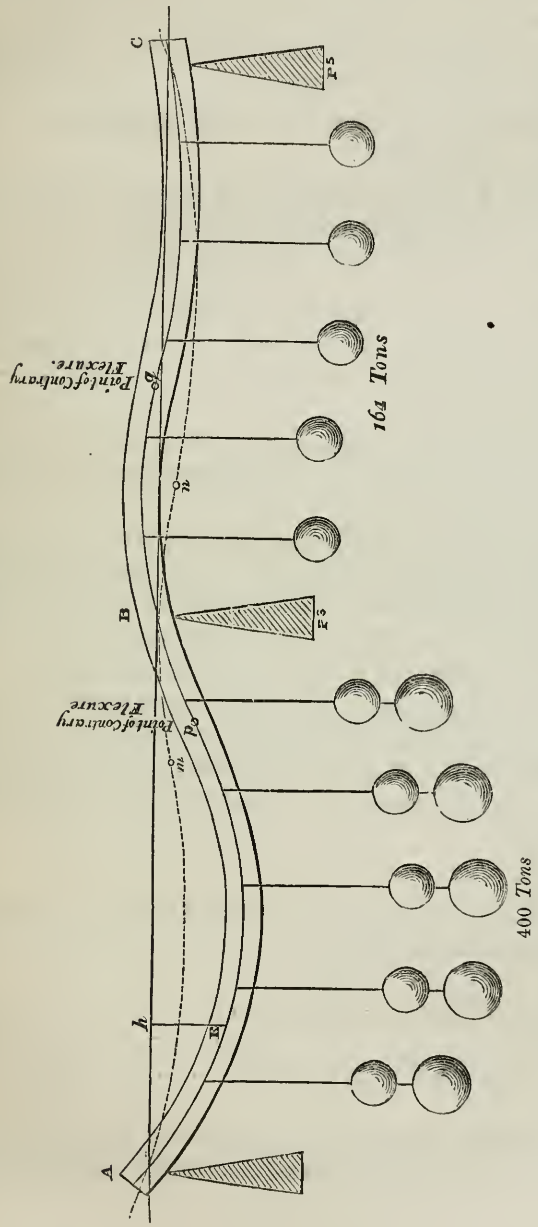
* This investigation is extracted from the 9th volume of the Proceedings of the Institution of Civil Engineers, 1851. Mr. Pole has given a more extended investigation in Mr. E. Clarke's 'Conway and Britannia Bridges.'

Fig. 95.



The dotted line represents the position of the girder as deflected by its own weight only, viz. 154 tons distributed over each span.

To determine the deflection curve of the beam, and the strength of the part A B.

Let l = A B or B C = the length of each opening.
μ = weight per lineal unit distributed over space A B.
Let μ_2 = Weight over space B C.
$\left.\begin{matrix} P_1 \\ P_3 \\ P_5 \end{matrix}\right\}$ = pressures upon the three supports A B and C respectively.
x = A h = horizontal distance from A of any point R in the neutral line of the beam.
y = h R = the deflection at that point.

If R be any point in the neutral line of the beam, whose co-ordinates are x and y, the portion R A of the beam is held in equilibrium by three forces, viz.:—

1st. The pressure P_1.

2nd. The load upon that portion of the beam $= \mu\, x$.

3rd. The elastic forces called into operation on the transverse section of the beam at R.

The principle of the equality of moments must therefore obtain in reference to these forces; *i. e.*, the sum of the moments of the 2nd and 3rd, which act together to turn the part R A round R in one direction, must be equal to the moment of the 1st, which tends to turn it in the opposite direction.

Now, 1st. The pressure P_1 acts at a perpendicular distance from R $= x$; therefore the moment of this force $= P_1\, x$.

2nd. The load $\mu\, x$ may be considered as collected at a point distant $\frac{1}{2}\, x$ from R; therefore the moment of this force $= \frac{\mu\, x^2}{2}$

3rd. Let, for the present, the moment of the elastic forces of the beam round R $= \phi$.

$$\text{Then } \phi + \frac{\mu\, x^2}{2} = P_1 x.$$

$$\text{or, } \phi = P_1\, x - \frac{\mu\, x^2}{2} \quad . \quad . \quad . \quad . \quad (1).$$

If E represent the *modulus of elasticity* of the beam, and I the moment of inertia of its transverse section round the neutral line, the moment of the elastic forces will be represented by the equation,

$$\phi = -\text{E I}\frac{d^2 y}{d x^2}.$$

whence we have

$$\text{E I}\frac{d^2 y}{d x^2} = \frac{\mu x^2}{2} - \text{P}_1 x \quad . \quad . \quad . \quad . \quad . \quad (2).$$

Integrating this, and representing the inclination to the horizon of the tangent to the neutral line at B by β, so that at that point, $\frac{d y}{d x} = \tan \beta$, we have

$$\text{E I}\left(\frac{d y}{d x} - \tan \beta\right) = \frac{\mu}{6}(x^3 - l^3) - \frac{\text{P}_1}{2}(x^2 - l^2)$$

Integrating again

$$\text{E I}(y - x \tan \beta) = \frac{\mu}{6}\left(\frac{x^4}{4} - l^3 x\right) - \frac{\text{P}_1}{2}\left(\frac{x^3}{3} - l^2 x\right) . \quad . \quad (3),$$

which is the equation to the deflection curve from A to B.

At the point B, when $x = l$, we know that $y = o$; therefore, by substituting these values in equation (3), we obtain,

$$\text{Tan}\, \beta = \frac{l^2}{24\,\text{E I}}(3 \mu l - 8\, \text{P}_1) \quad . \quad . \quad . \quad (4).$$

Now, by applying a similar process to the part B C of the beam, and remembering that the angle β must in this case have a contrary sign, we obtain

$$\text{Tan}\, \beta = \frac{l^2}{24\,\text{E I}}(8\, \text{P}_5 - 3 \mu_2 l) . \quad . \quad . \quad . \quad (5).$$

Comparing this with equation (4) we obtain

$$3 \mu l - 8\, \text{P}_1 = 8\, \text{P}_5 - 3 \mu_2 l . \quad . \quad . \quad . \quad . \quad (6).$$

By the principle of equality of moment round B, we have

$$\text{P}_1 l + \frac{\mu_2 l^2}{2} = \text{P}_5 l + \frac{\mu l^2}{2}, \quad . \quad . \quad . \quad . \quad . \quad (7),$$

whence by substitution with equation (6)

$$P_1 = \frac{7\,\mu\,l - \mu_2\,l}{16},^* \quad . \; . \; . \; . \; . \; . \; . \; . \quad (8)$$

$$\text{and } P_5 = \frac{7\,\mu_2\,l - \mu\,l}{16}, \quad . \; . \; . \; . \; . \; . \; . \quad (9)$$

and since $P_1 + P_3 + P_5 = \mu\,l + \mu_2\,l$,

$$P_3 = \frac{5}{8}\,(\mu_2\,l + \mu\,l) \; . \; . \; . \; . \; . \; . \; . \; . \quad (10).$$

To find the point of contrary flexure in the curve $A\,R\,B$; or where

$$\frac{d^2\,y}{d\,x^2} = 0.$$

Referring to equation (2), we have $0 = \frac{\mu\,x^2}{2} - P_1\,x$,

$$\text{or } x = \frac{2\,P_1}{\mu} \text{ at the point of contrary flexure.}$$

It is evident that at this point $\phi = 0$, i.e., there are no elastic forces exerted, and therefore there are no longitudinal strains, either of extension or compression, on any of the fibres of this section of the beam.

We may now proceed to calculate the strength of the part A B of the beam; and this resolves itself into the question, What is the greatest longitudinal strain on the fibres of the beam, when bearing a given load?

Let the load distributed over the length $A\,B = \mu\,l$ as before.

Now, in order to find the place in this length where there is the greatest strain on the fibres, or where ϕ, the moment of the elastic forces, is at a maximum, differentiate equation (1) and make $\frac{d\,\phi}{d\,x} = 0$.

We have thus

$$0 = P_1 - \mu\,x, \text{ or } x = \frac{P_1}{\mu} \quad . \; . \; . \quad (12)$$

at the place of greatest strain. This, it will be observed from equation (11), is half-way between the end of the beam and the point of contrary flexure.

Substituting between equations (1), (8), and (12), we have

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[*] If $\mu = \mu_2$, *i.e.*, if the load is equal on both sides of the centre pier,

$$P_1 = P_5 = \tfrac{3}{8}\,\mu\,l,$$
$$P_3 = \tfrac{10}{8}\,\mu\,l.$$

for the moment of elastic forces at the section of greatest strain,

$$\phi = \frac{(7\,\mu\,l - \mu_2\,l^2)}{512\,\mu} \quad . \quad . \quad . \quad . \quad . \quad . \quad (13).$$

It is capable of proof, that if f = the longitudinal strain, per square unit of area, on any fibre of the beam; c = distance of that fibre from the neutral line; and I = the moment of inertia of the section of the girder round the neutral line; then

$$\text{Moment of elastic forces} = \phi = \frac{f}{c}\,\text{I}.$$

Therefore, by equation (13), at the section of greatest strain,

$$\frac{f}{c}\,\text{I} = \frac{7\,\mu\,l - \mu_2\,l^2}{512\,\mu}$$

$$\text{or } f = \frac{c\,(7\,\mu\,l - \mu_2\,l^2)}{512\,\text{I}\,\mu}$$

which by using the proper value of c will give the greatest strain either of extension or compression, on any of the fibre of the beam, and will thus determine the *strength* of the beam to resist a given load.

Application to the Torksey Bridge.

The following are the values of the given quantities for the case in question:—

l = clear span = 1560 inches.

$\mu\,l$ = load on A B = 400 tons, or for each girder = 200 tons.

$\mu_2\,l$ = load on B C = 164 tons, or for each girder = 82 tons.

E = modulus of elasticity, is taken at 10,000 tons* for a bar one inch square.

To find the position of the neutral line.

It is known that when the material of a beam is perfectly elastic, the neutral axis of any transverse section passes through its centre of gravity.

By the application of this rule to the section of the Torksey Bridge girders, the neutral line is found 64 inches from the top or 56 inches from the bottom of the section.

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* The value used for the deflection of the Britannia and Conway Tubular Bridges.

To find the moment of inertia I *of the transverse section round its neutral axis.*

Since we have $I = \Sigma \rho_2 \Delta k$, the moment of inertia is obtained by adding together the moments of all the separate parts of the section. The moments of the horizontal plates are found by simply multiplying the area of each by the square of its vertical distance from the neutral line; those of the vertical plates by the application of well-known analogous rules. The following are the results derived from two independent computations. The dimensions are taken in inches.

Moment of inertia of the section of the girder round the neutral line.

Compressed portion.	
Top plates	73,700
Vertical plates of cells . .	41,700
Bottom plates of cells . .	51,700
Portion of side plates . .	21,100
Total moment of compression	188,200
Extended portion.	
Portion of side plates . .	28,500
Bottom plates	155,800
Total moment of extension .	184,300
Total sum of moments = I .	372,500

The values of the pressures on the three points of support are obtained from equations (8), (9), and (10). They are for each girder,

$P_1 =$ . . . . 82·375
$P_5 =$ . . . . 23·375
$P_3 =$ . . . . 352·500

The value of $\tan \beta$ (β being the angle the girder makes with the horizontal) is obtained from equation (4):

$$\tan \beta = -0{\cdot}0014955$$

The distance of the point of contrary flexure from A, is, by equation (11), = 1285 inches, or 22 feet 11 inches from the centre pier.

The deflection of the loaded span of the beam is obtained by equation (3), and that of the unloaded span by one similarly deduced. The deflection of the unloaded beam may be found in a corresponding manner, and all three are united in the following table:—

Calculated Deflection of the Torksey Bridge under the specified Load.

Distance from end.		Deflection of loaded girders.	Deflection of girders from their own weight.	Deflection due to load.
	feet.	inches.	inches.	inches.
End Pier A. .	.	0·00	0·00	0·00
Loaded Span .	10	+0·41	+0·13	+0·28
	20	0·79	0·25	0·54
	30	1·12	0·35	0·77
	40	1·36	0·42	0·94
	50	1·50	0·45	1·05
	60	1·55	0·45	1·10
	70	1·49	0·42	1·07
	80	1·34	0·35	0·99
	90	1·11	0·27	0·84
	100	0·83	0·18	0·65
	110	0·53	0·10	0·43
	120	+0·24	+0·03	+0·21
Centre Pier B. .	.	0·00	0·00	0·00
Unloaded Span .	120	−0·14	+0·03	−0·17
	110	−0·20	0·10	−0·30
	100	−0·21	0·18	−0·39
	90	−0·18	0·27	−0·45
	80	−0·12	0·35	−0·47
	70	−0·06	0·42	−0·48
	60	−0·01	0·45	−0·46
	50	+0·04	0·45	−0.41
	40	+0·07	0·42	−0·35
	30	+0·07	0·35	−0·28
	20	+0·06	0·25	−0·19
	10	+0·02	+0·13	−0·11
End Pier C. .	.	0·00	0·00	0·00

The greatest longitudinal strain on the fibres of the beam is determined from equation 14, as follows:—

To find the greatest compressive strain on the top plates, we must make $c = 64 =$ the distance of these plates from the neutral line; whence $f = 4·55$ tons per square inch, greatest longitudinal compressive strain on the top plates.

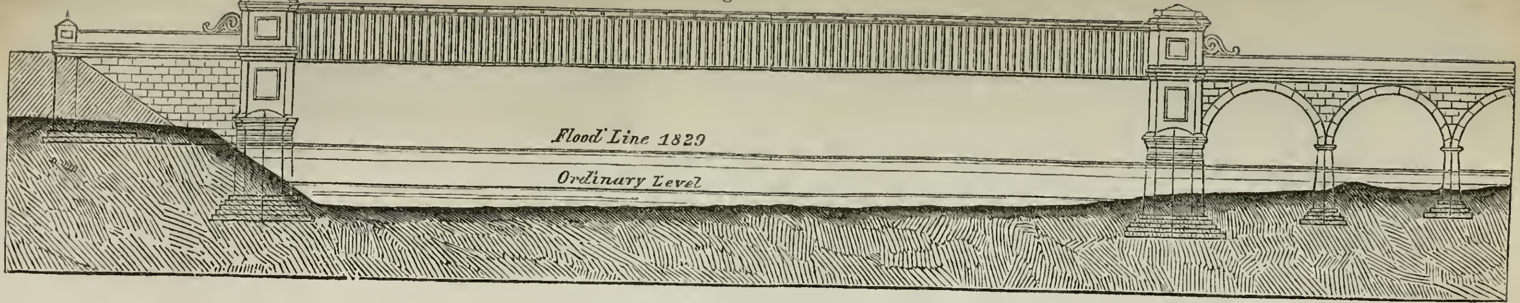
For the bottom plates $c = 56$, whence $f = 4$ tons per square inch, greatest longitudinal tensile strain on the bottom plates.

The advantage gained by the continuity of the girder across the two openings is shown by the following table. The first column applies to the continuous girder, the second contains the corresponding strength and deflection calculated for an independent girder spanning one opening only. It will be seen that the effect of the continuity is to increase the strength in the ratio of about 3 : 2, and to diminish the deflection in a still larger proportion.

	In the continuous girder.	In an independent girder, spanning one opening only.
	Tons per square inch.	Tons per square inch.
Compressive strain on top plates . .	4·55	6·75
Tensile strain on bottom plates . .	4·00	5·90
	inches.	inches.
Deflection by weight of structure only .	0·45	1·08
Deflection with load added . . .	1·55	2·65

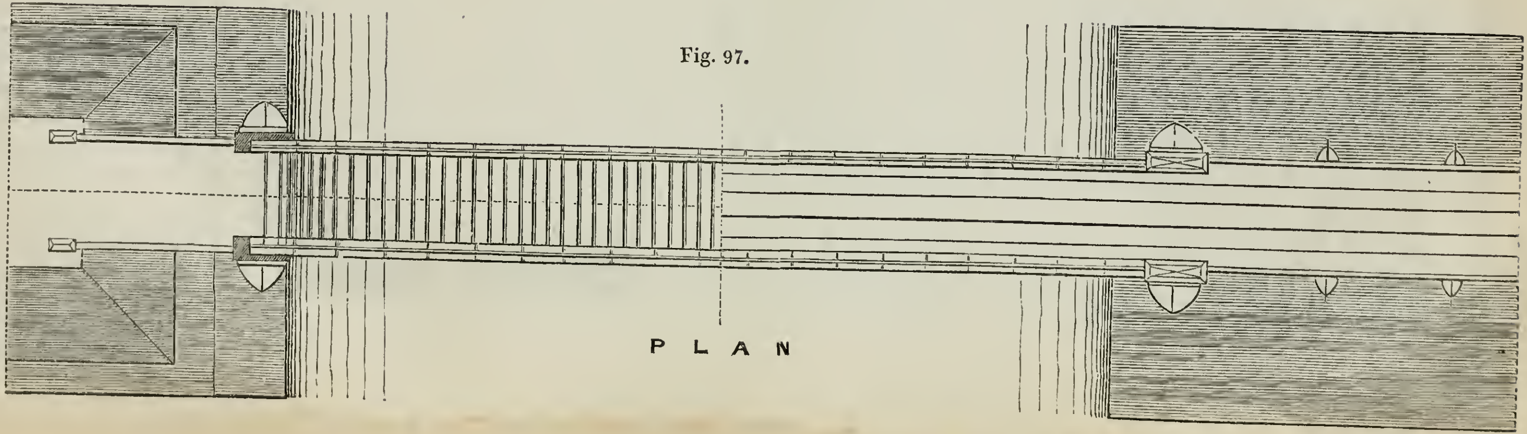
I have given the above investigation of Mr. Pole rather to stimulate experimental inquiry than to imply reliance in his conclusions, as a rule for the construction of tubular girder bridges. Mr. Pole, it will be observed, attempts to prove that the strength of a continuous girder like that of the Torksey Bridge, extending over two spans, is to that of an independent girder as 3 : 2. Now the formula for this calculation may or may not be correct as the premises on which it is founded approach or recede from the truth; it is apparently, however, not derived from direct experiment, but from assumed data, which may be questioned in our attempts to reduce it to practice. There cannot, however, exist a doubt that in loading a continuous beam in the middle of any one span, the tendency will be to throw the top part of the beam immediately over the pier into a state of tension, and that tension cannot be produced without a corresponding decrease of deflection in the beam, and a tendency to raise the middle part of the beam of the adjoining span in the ratio of that force. These conditions may safely be taken into account in the calculations of continuous beams, but I would recommend the exercise of caution in trusting to theoretical formulæ, which in general practice might

Fig. 96.



ELEVATION

Fig. 97.



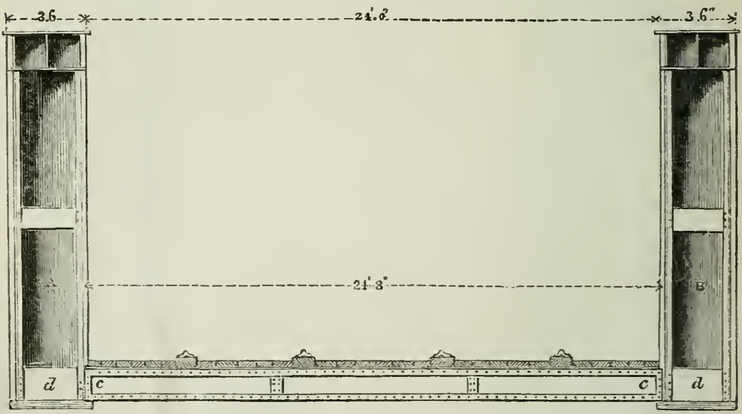
PLAN

lead to results unfavourable to the strength and security of these important structures.

As already stated, I have purposely omitted, in my calculations, taking into account those elements of strength which peculiarly belong to the continuous beam; I have done so, not from any desire to lessen its importance, but to guard the practical builder against an allowance of one-third in the strength of the continuous girder. A reduction to that extent I should consider unsafe in general practice; and taking into account the various forms and conditions under which girders of this description are constructed, I should consider it much more secure —without questioning the accuracy of Mr. Pole's formula,— to limit the reduction, under such circumstances, to one-fifth.

On the construction of tubular girder bridges of single span, I have selected for illustration an example which may serve to show how these bridges are constructed, and how the different parts are united so as to give the required tenacity and the necessary rigidity to the force and velocity of a railway train. This bridge supports the Inverness and Aberdeen Junction Railway across the Spey, Joseph Mitchell, Esq., Engineer, and consists of a single span of 230 feet in the clear, as will be seen by reference to the elevation and plan, figs. 96 and 97. It consists of two wrought-iron tubular girders, each 16 feet deep, 3 feet 6 inches wide. Each girder is of sufficient length, 245 feet, to

Fig. 98.



give a bearing surface of 7 feet 6 inches on the piers at each end. It is supported on one side upon cast-iron bed-plates, and at the other upon rollers, which leave it free to expand and contract under all the changes of atmospheric temperature. Between the tubes the railway is supported upon longitudinal balks of timber resting on wrought-iron plate beams at every 4 feet, riveted to the sides of girders, as may be seen at *c c* in the section. Fig. 98 exhibits a cross section of the bridge at the centre, and shows the relative positions of the tubular girders A and B, placed at a distance of 24 feet 8 inches apart. One of the wrought-iron cross beams is seen at *c c* supporting the longitudinal planking of the roadway, and connected to the sides by the plates *d d*, which extend the bearings to the outside of each girder.

The Top. The arrangement of the plates in each girder will be seen in fig. 99, which represents a cross section of one of them at the centre. The top of the girder is formed on the cellular principle, calculated to present an adequate power of resistance to the compressive strain to which the top is subjected; and to prevent the *buckling* to which thin wrought-iron plates are liable. The cells A B, fig. 99, two in number, are composed of plates, each $\frac{8}{16}$ inch thick, and 1 foot 9 inches wide, and of three vertical or V plates $\frac{7}{16}$ inch thick and 1 foot 6 inches wide. These plates are connected together by eight angle-irons, each 4 inches by 4 inches by $\frac{9}{16}$ inch running longitudinally the entire length of the girder, and riveted to the plates at every three inches. The joints in the plates are covered with strips on each side, riveted with two rows of rivets, that is, one row in each plate. Fig. 100, page 257, is a plan of the top of the girder, showing the strips covering the longitudinal and cross joints in the A plates.

The Sides are composed of S plates $\frac{5}{16}$ inch thick at the centre and 2 feet wide, covered on the outside of the tube by strips, and on the inside by T irons in the manner shown in fig. 99, in order to give rigidity to the construction of those parts and to retain the top and bottom in position. Towards the ends, T irons take the place of the covering strips on the outside, and the bottom is widened and connected with the sides by triangular *gussets* (fig. 102). To prevent the buckling of the sides, small plates marked G, fig. 99, are introduced at every two feet and riveted to the T irons on each side; and at the

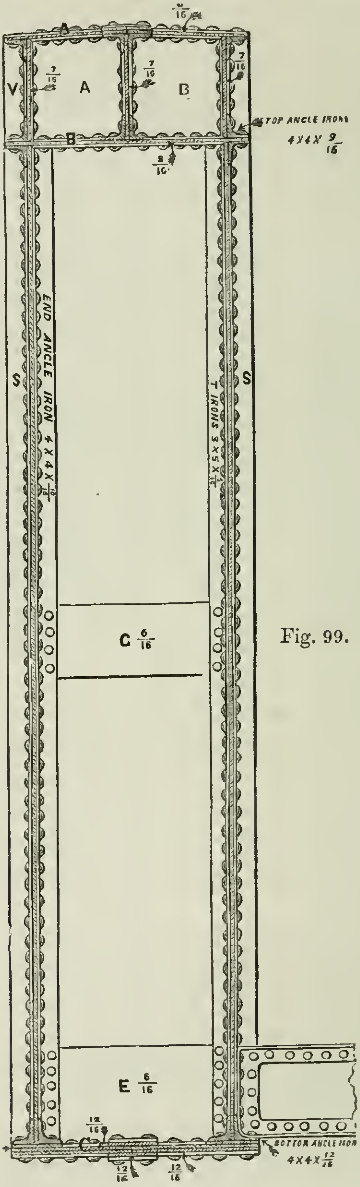
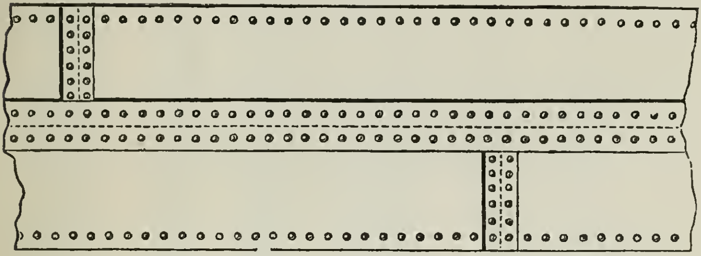


Fig. 99.

bottom to prevent distortion, and to throw the strain of the cross beams on the centre of the girders, similar plates marked E are inserted.

Fig. 100.



The bottom is not of the cellular construction; indeed, where sufficient sectional area can be obtained without cells, the more nearly the bottom approximates to a solid homogeneous mass, the better is it calculated to resist a tensile strain. In this girder it is composed of large and heavy plates 12 feet long, 1 foot 9 inches broad, and $\frac{12}{16}$ inch thick, of which four are seen in the cross section at the centre; these are placed with the joints alternating, and covered with plates 2 feet 8 inches long, carefully chain-riveted. The bottom is connected with the side plates by two large angle-irons 4 inches × 4 inches × $\frac{14}{16}$ inch, a packing strip being interposed to secure a level surface for the ends of the S plates, uninterrupted by the covering plates on the top of the C plates. Strips 8 inches broad cover the longitudinal joint between the C plates, and the whole is united into an almost homogeneous mass by rivets 1 inch in diameter placed 4 inches apart. Fig. 101 shows a plan of the girder, bottom upwards, with the position of the covering plates, &c., and also a section through the line *a b*, with the covering plates alternating so as to break the joints.

On one abutment the girders rest simply upon a cast-iron bed-plate firmly fixed in the masonry, but the other on rollers to permit free longitudinal motion as the girders expand or contract with the changes of temperature. In the Spey Bridge the variation will not probably exceed $1\frac{3}{4}$ inch, but in the Britannia Bridge it amounts to as much as a foot. Fig. 102 is a cross section, and fig. 104 an outside elevation of part of one of the girders at the abutment, showing the position and arrange-

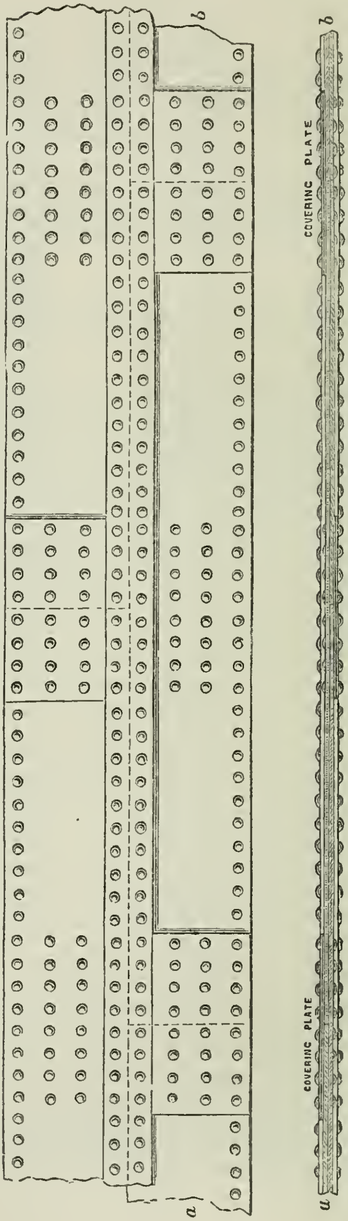


Fig. 101.

ment of the rollers. The cast-iron bed-plate (*b b*) upon the abutment is 8 feet long, 5 feet broad, and 2 inches thick,

Fig. 102.

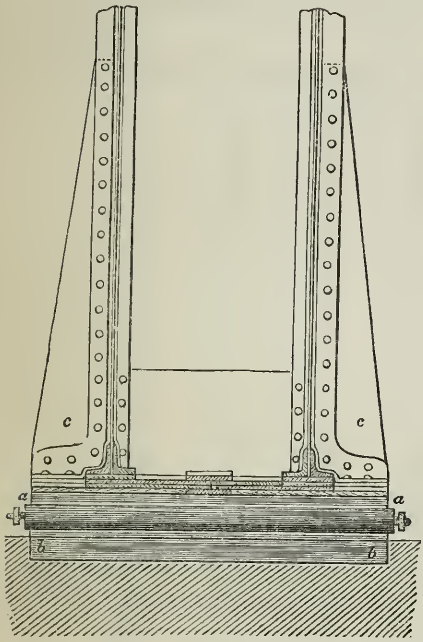
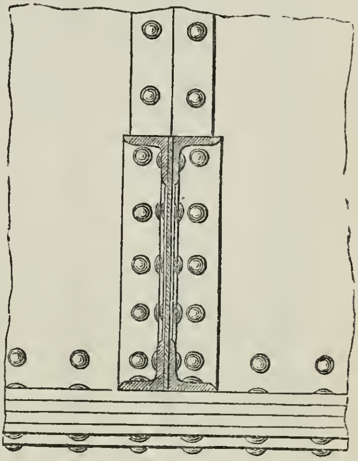


Fig. 103.

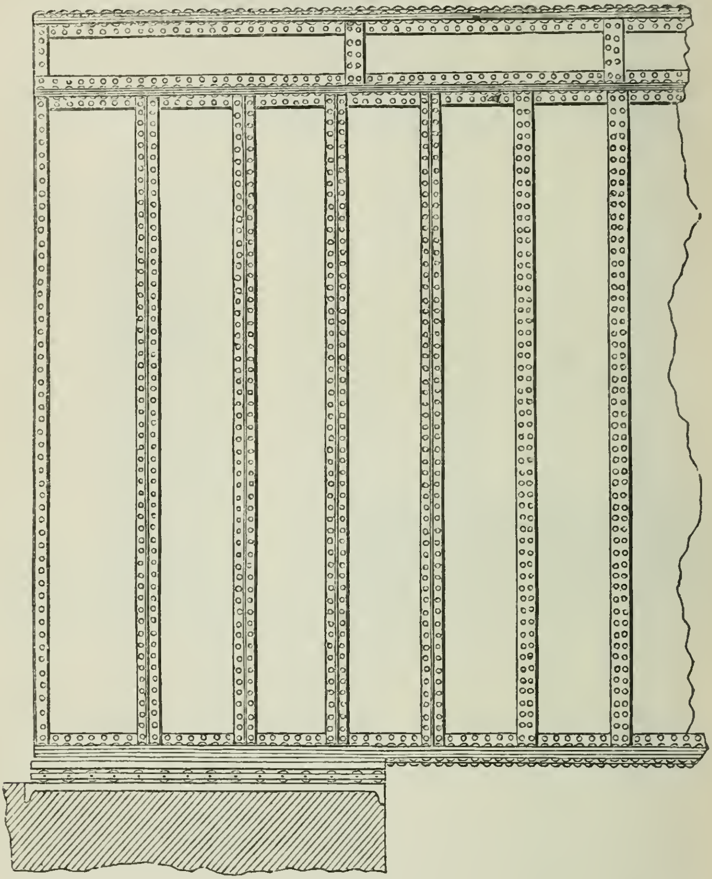


with flanges at each end for holding it in the masonry. That under the girder is similar, but without flanges, and only 7 feet 8 inches in length. The girder is fitted upon the latter with thin strips of wood dipped in tar so as to obtain a firm and even bearing surface. Between the bed-plates are placed the rollers, 12 in number, and each 4 inches in diameter and 5 feet 2 inches in length; the ends are turned down into axes which are fixed in a rectangular wrought-iron frame to keep them parallel to one another and to the end of the girder.

Fig. 102 also shows the manner in which the bottom of each girder is expanded to a breadth of 5 feet over the whole of the bearing surface, the T irons being bent round, the gussets *c c* are riveted to them to increase the width of the base and give increased lateral rigidity to the part which rests upon the masonry.

The cross beams which support the roadway are of wrought iron like the bridge itself; they consist of a vertical plate of wrought iron (fig. 103) 1 foot 4 inches deep, and $\frac{5}{16}$ inch thick, to the top and bottom of which are attached four angle-irons, to form the flanges of the beam. These angle irons are each 3 inches by 3 inches by $\frac{1}{2}$-inch. At the ends the angle-irons are bent round and riveted to the beam and to the side of the girder. These cross beams are placed at distances of 4 feet apart.

Fig. 104.



It is to be observed that the thicknesses of the plates, as given above, apply only to the centre of the bridge; towards the ends the top and bottom plates gradually diminish while the side plates slightly increase in thickness as they approach the piers or abutments of support. This arrangement is made to proportion the different parts of the girder to the strains they have respectively to bear.

In calculating the ultimate strength of this bridge, we may take the heaviest rolling load which could ever come upon it, viz. both lines being covered with locomotives, or two heavily loaded goods trains, drawn by four engines, passing, one on the up, the other on the down line at the same time, to be equivalent to $1\frac{1}{2}$ ton per lineal foot $= 230 \times 1\frac{1}{2} = 345$ tons, distributed over the two lines of rails in the distance between the abutments of the bridge.

Taking the sectional area of the bridge, we have—

Area of Top of one Girder.

			Sq. in.
4	A and B plates, $21 \times \frac{8}{16}$ . . .	=	42·0
2	V plates, $18 \times \frac{7}{16}$	=	15·7
1	V plate, $18\frac{3}{4} \times \frac{7}{16}$	=	8·2
10	Angle-irons, $4 \times 4 \times \frac{9}{16}$ . . .	=	41·8
	Covering strip, $8 \times \frac{8}{16}$ . . .	=	4·0
	Total sectional area . .	=	111·7

Area of Bottom of one Girder.

			Sq. in.
4	C plates, $21 \times \frac{12}{16}$	=	63·0
2	Strips, $8 \times \frac{12}{16}$	=	12·0
4	Angle-irons, $4 \times 4 \times \frac{14}{16}$ . . .	=	25·2
	Total sectional area . .	=	100·2

Hence, in the formula $W = \frac{a\,d\,c}{l}$, we have a = area of the bottom $= 100·2$ square inches; d = depth of girder $= 16 \times 12 = 192$ inches; $c = 80$; l = length of span $= 230 \times 12 = 2760$ inches.

Hence, by formula—

Tons.

$\frac{100{\cdot}2 \times 192 \times 80}{2760} = 557{\cdot}6 =$ Centre breaking weight of one main girder.

$\therefore\ 557{\cdot}9 \times 2 = 1115{\cdot}2 =$ Centre breaking weight of bridge.

$\therefore\ 557{\cdot}6 \times 4 = 2230{\cdot}4 =$ Breaking weight of bridge load equally distributed.

Or in other words $9{\cdot}6 =$ Breaking weight per lineal foot.

Hence ratio of breaking weight to greatest load after deducting for the weight of the bridge $= 8{\cdot}1 : 1{\cdot}5$ or $5{\cdot}4 : 1$.

The bridge is estimated to weigh about 350 tons.

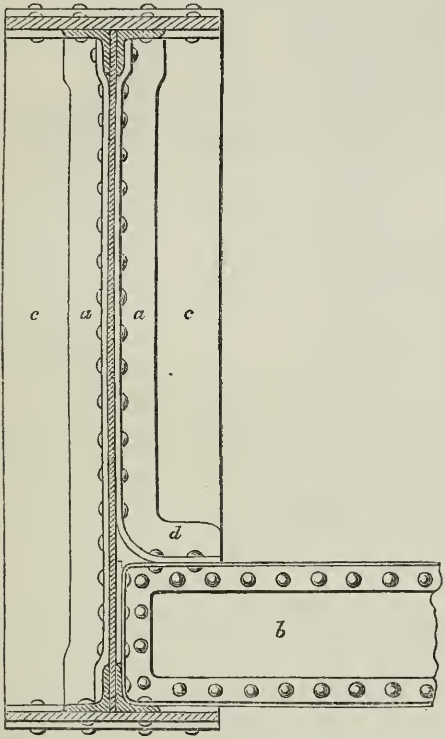
Plate Bridges.

For small spans not exceeding 60, 80, or 100 feet, the tubular arrangement is frequently laid aside, and the girder is constructed in the form of a simple beam. I have already alluded to this point, and have only now to add an example of the manner in which plate beams are applied in bridge-building.

The great cause affecting the durability of iron bridges is the oxidation which arises from a damp atmosphere, especially in those crossing tidal rivers, arms of the sea, &c., when saline particles are held in suspension, and where they are exposed to the alternate changes of wet and dry; in such situations, if precautions were not taken, there is reason to believe that this exposure would in time be productive of serious if not disastrous results. To obviate any danger from this cause tubular and tubular girder bridges are designed, so that access may be gained to every part for the purpose of painting. Thus, for instance, the cells of the Britannia Bridge and of Spey Bridge are sufficiently large to allow a man or a boy to push himself through them on a small truck. But in bridges of small span this cannot always be provided for, and hence the superiority of a plate girder. Simplicity of construction and cheapness are also great advantages of this form, which more than compensate for some slight loss of strength.

Fig. 105 is a cross section of one girder of a plate bridge of 55 feet 9 inches span, designed to carry a single line of rails. The bridge consists of two girders, each 62 feet long and 5 feet deep, placed at a distance of 12 feet 9 inches apart between the girders, which in this construction as in that of the tubular girders form the parapets of the bridge. The plates forming the top are 12 feet and 14 feet long, 18 inches broad, and varying from $\frac{12}{16}$ inch in thickness at the centre to $\frac{10}{16}$ at the ends. The joints are covered by strips. The bottom is composed of similar plates, varying in thickness from $\frac{10}{16}$ at the centre to $\frac{8}{16}$ at the ends, carefully chain-riveted; the sides are composed of plates 4 feet $10\frac{5}{8}$ inches long, by 2 feet broad, and $\frac{6}{16}$ inches thick; except the two last plates at each end, which are $\frac{7}{16}$ inch thick. The joints in these plates are covered alternately by strips 5 inches broad and by ⊤ irons (*a a*) $4\frac{1}{2}$ inches × $3\frac{1}{2}$ inches × $\frac{10}{16}$ inches riveted to the plates on each side; the ⊤ irons being placed at those joints at which the cross beams are riveted. The top and bottom plates are connected with the side plates by four angle-irons 4 inches × 4 inches × $\frac{10}{16}$ inches riveted to each. The ends of the girders are rendered firm and rigid by the large plate shown at *c c*. The roadway of planking

Fig. 105.



is supported upon cross beams (*b*) of wrought-iron resting on the bottom flange of the girder and riveted to the side plates; to give still further strength and rigidity in this bridge, the inside T irons are not continued to the bottom of the girder, but are bent round at *d* so as to clasp the tops of the cross beams, to which they are riveted.

As these are not quite so strong as the tubular girders, the constant c, in the formula

$$\mathrm{W}=\frac{a\,d\,c}{l},$$

is taken = 75 instead of 80 as before.* Hence in this bridge we have:—

Sectional Area of Top.

		Sq. in.
One plate, 18 inches $\times \frac{14}{16}$	=	16·75
Two angle-irons, $4 \times 4 \times \frac{10}{16}$	=	9·20
		25·95

Sectional Area of Bottom.

		Sq. in.
One plate, $18 \times \frac{10}{16}$	=	11·2
Two angle-irons	=	9·2
		20·4

Tons.

Hence $\mathrm{W} = \dfrac{20{\cdot}4 \times 60 \times 75}{669} = 137$ = Centre breaking weight of one girder.

Or $137 \times 2 = 274$ = Centre breaking weight of bridge.

And again, $137 \times 4 = 548$ = Breaking weight of bridge, load equally distributed over the surface of the bridge.

Which is equivalent to 9·9 = Breaking weight per lineal foot.

Hence, ratio of breaking weight to greatest load is as 9 : 1, when 40 tons is deducted for the weight of the bridge.

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* This diminution of strength does not arise from any deficiency in the areas of the top and bottom plates of the girder, but from a want of lateral stiffness in the single-plate construction as compared with that of the tubular form.

Figs. 106, 107, exhibit two additional forms of plate girders with cellular tops. They are simply modifications of the tubular and plate forms, and, although admirably adapted for securing the greatest resistance to the force of compression, they are nevertheless objectionable on the score of corrosion, which occasionally occurs in the interior of the cells unless made of capacity sufficient to admit of their being cleaned and painted.

Fig. 106.

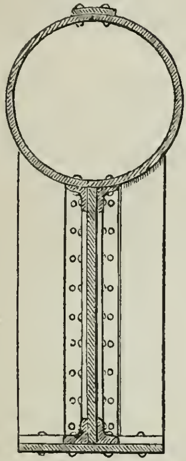
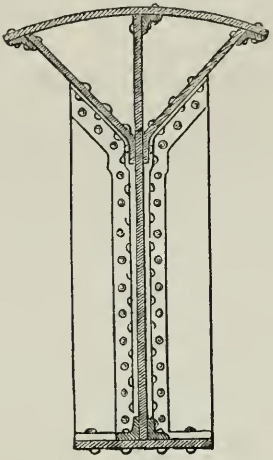


Fig. 107.



From the time of the experiments which determined the form and construction of the large tubular bridges may be dated the introduction of wrought-iron beams and wrought-iron girder bridges of almost every description. Before that date, (1845-6,) our knowledge of the properties of wrought iron, and its application to the useful arts, was very imperfect. It had been used in the construction of boilers, steam engines, and water wheels, from a comparatively early period, and even at that time, and for some years previous, it was making rapid progress in its application to ship-building. Its properties, distribution, and appliance to beams and bridges, were, however, unknown and unappreciated until the experiments referred to proved its superiority over every other material then known for the attainment of objects for which it has since been so largely and so extensively in demand. As a material for the construction of bridges, wrought iron was universally condemned, and some of our ablest mathematicians went so far as to prove its inefficiency in the shape of rectangular tubes composed of riveted plates, as being perfectly utopian, and, to employ the expression then made use of, '*It would crumple up like a piece of leather!*'

In opposition to these prognostications we have lived to see the contrary, and to witness the wide and universal extension of a principle now recognised as one of the most important to the advancement and extension of the railway system.

Among the numerous constructions founded upon the experiments referred to, we have those of Mr. Brunel, as observed in many of them peculiar to himself. Mr. Brunel was among the first to avail himself of the improvements then suggested, and he was not slow to introduce into his constructions such forms and material as in those which span the Wye at Chepstow, and the greatly enlarged structures now in progress for supporting the Cornish Railway across the tidal estuary at Saltash near Plymouth. Mr. Brunel has moreover availed himself of the cellular system in the construction of the large ship the 'Great Eastern,' and that principle has been applied with certainty and effect in affording greatly increased strength and security to that large and important structure.

Other bridges besides those of Mr. Brunel, as exhibited at Chepstow and Saltash, have been introduced by Messrs. Fox and Henderson, and others. Taking for example the Bowstring Bridge, in which the resisting powers are in the arch from which the roadway is suspended, and in the chord line of riveted plates, which have been successfully introduced on the principle of chain-riveting and the stiffening of the platform as a means of resistance to the tensile strain and the retention of the bridge in form. To attain the required rigidity in the arch, Messrs. Fox and Henderson have introduced the tubular or cellular system, and by a judicious application of plates and angle-iron the required stiffness and stability is attained. As a whole, the Bowstring bridges may be said to approximate to those of Mr. Brunel, or vice versâ—as the one is supported or suspended from rectangular tubular arches, and the other from a curved circular tube supported by rigid frames of cast or wrought iron at each end; and from these again is suspended the roadway, composed of plate girders, by strong diagonal and vertical bars of iron. Altogether the principles of these two constructions are not widely different, and so far they have successfully answered the purposes for which they were intended.

Attempts have been made, and are now making, to connect the chain suspension bridge into an unyielding, rigid structure, which to some extent is successful; but I would respectfully caution my professional contemporaries against those constructions, as I think the principle is unsound and calculated to lead to disastrous and unfortunate results. It is true that the American engineers have contrived to render the Wire Bridge at Niagara sufficiently rigid to admit of a railway train crossing at a slow speed; but I would refer the reader to the experiments on trussed girders, (page 41,) to show that the suspension of a stiffened frame or a rigid girder is not the best form in which to apply the material with a view to obtain strength. On the contrary, we have proved that a flexible material, when united to one that is perfectly rigid, is a dangerous construction, and that it is almost next to impossible for them to work together without endangering the security of the structure. It has been shown that the tension of the bars or chains forming the truss are seldom if ever in unison with the bearing powers of the beam it is called upon to support; in some positions of the passing load they are antagonistic to each other, and hence follows that overpowering and unequal strain which, often repeated, terminates in the destruction of the bridge.

If a bridge of this description be the only one calculated to meet the exigencies of any particular case, let it be made double the strength, and in doing so it will then become a question whether it would not have been preferable to have used the same amount of material in another form, and given to the construction all the rigidity and strength that is so simply and effectually obtained in the form of the common girder.*

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* Mr. Barlow brought this subject before the mechanical section of the British Association for the Advancement of Science, held in Dublin in August last; and although his designs for the bridge intended for crossing the Foyle at Londonderry were ingeniously contrived, it will however become a subject of mature reflection,

In these very important constructions we could give a greatly increased number of examples, both as regards form and the objects to which they are applied ; but having already exceeded the limits assigned for this department of inquiry, we are compelled to forego further description, and confine ourselves to a short account of the tubular and tubular girder bridge as submitted to the Prussian Government for adoption in 1850 and 1851.

Although this work is intended to be almost exclusively practical, I must not, however, omit to give a short account of a paper *On the Law of Strains on Beams*, by the Astronomer Royal. In this paper the mathematical investigations and line of strains are so ably demonstrated and set forth in the diagrams as sufficient to warrant their introduction in this place, and having been presented with a copy, with the permission of the author to make what use of it I please, I am now enabled—viewing the question in its practical as well as theoretical bearings—to give the following illustration of the theory which cannot fail—in connection with the diagrams—to be otherwise than useful to the mathematician and the general reader.

On the Strains in the Interior of Beams.

By George Biddell Airy, *F.R.S., Astronomer Royal.*

The author states that he had long desired to possess a theory which should enable him to compute numerically the strains on every point in the interior of a beam or girder, but that no memoir or treatises had given him the least assistance.* He had therefore constructed a theory which solved completely the problems for which he wanted it, and which appears to admit of application at least to all ordinary cases.

The theory contemplates forces acting in one plane. A beam, therefore, is considered as a lamina in a vertical plane, the same

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on the part of Mr. Barlow, whether a decidedly stronger and more effective principle can or cannot be introduced by a judicious application of the same quantity of material in a different form and at less cost.

* Subsequently to the communication of this Report, the author learned that one instance (the second) of those given here had been treated by Professor Rankine, by methods peculiar to that instance.

considerations applying to every vertical lamina of which a beam may be conceived to be composed.

The author remarks that it is unnecessary to recognise every possible strain in a beam. Metallic masses are usually in a state of strain, from circumstances occurring in their formation; but such strains are not the subject of the present investigation, which is intended to ascertain only those strains which are created by the weight of the beam and its loads. The algebraical interpretation of this remark is, that it is not necessary to retain general solutions of the equations which will result from the investigation, but only such solutions as will satisfy the equations.

After defining the unit of force as the weight of a square unit of the lamina, and the measure of compression-thrust or extension-pull as the length of the ribbon of lamina, whose breadth is the length of the line which is subject to the transverse action of the compression or tension, and whose weight is equal to that compression or tension, the author considers the effect of tension, &c., estimated in a direction inclined to the real direction of the tension, and shows that it is proportional to the square of the cosine of inclination. He then considers the effect of compounding any number of strains of compression or tension which may act simultaneously on the same part of a lamina, and shows that their compound effect may in every case be replaced by the compound effect of two forces at right angles to each other, the two forces being both compressions or both tensions, or one compression and one tension. Succeeding investigations are therefore limited to two such forces.

Proceeding then to the general theory of beams, it is remarked that if a curve be imagined, dividing a beam into any two parts, the further part of the beam (as estimated from the origin of coordinates) may be considered to be sustained by the forces which act in various directions across that curve, taken in combination with the weight of the further part of the beam, the load upon that part, the reaction of supports, &c. Expressing the forces in conformity with the principles already explained, and supposing that there is one compression-force B making an angle β with y (in the direction of y diminishing for increase of x), and another compression-force C making an angle $90 + \beta$ with y, it is easily seen that the element δs of the curve, supposed to make the angle θ with y, sustains the forces.

In x, $\quad \text{B} \,.\, \delta s \times \sin(\beta+\theta) \times \sin\beta + \text{C} \,.\, \delta s \times \sin(\beta + 90^\circ + \theta) \times \sin(\beta + 90^\circ)$.

In y, $-\text{B} \,.\, \delta s \times \sin(\beta+\theta) \times \cos\beta - \text{C} \,.\, \delta s \times \sin(\beta + 90^\circ + \theta) \times \cos(\beta + 90^\circ)$.

The weight of lamina is bounded by y and $y + \delta y$, and estimated as acting upwards, is $-y\delta x$. And the reaction R of a support may act upwards at distance h.

Expanding the sines and cosines, putting δx for $\sin\theta \,.\, \delta s$, and δy for $\cos\theta \,.\, \delta s$; putting also

$$\text{L} = \text{B} \,.\, \sin^2\beta + \text{C} \,.\, \cos^2\beta,$$
$$\text{M} = (\text{B} - \text{C}) \,.\, \sin\beta \,.\, \cos\beta,$$
$$\text{Q} = -\text{B} \,.\, \cos^2\beta - \text{C} \,.\, \sin^2\beta,$$
$$\text{O} = y - \text{Q},$$
$$p = \frac{dy}{dx},$$

and forming the equations of equilibrium in the usual way, they will be found to be—

Equation for forces in x, $\int dx \,.\, (\text{L}p + \text{M}) = 0$.

Equation for forces in y, $\int dx\,(\text{M}p + \text{O}) - \text{R} = 0$.

Equation of momenta, $\int dx(\text{L}yp + \text{M}y + \text{M}xp + \text{O}x) - \text{R}h = 0$.

Now these equations, applying to any curve, will apply to any two curves very close together; and therefore their variation, taken by the rules of the Calculus of Variations, will be 0. The proper equation (in the usual notation) is $\text{N} - \frac{d(\text{P})}{dx} = 0$. Applying this, the results are

$$\frac{d\text{M}}{dy} - \frac{d\text{L}}{dx} = 0,$$
$$\frac{d\text{O}}{dy} - \frac{d\text{M}}{dx} = 0.$$

From this it follows that (omitting some arbitrary functions which represent original strains in the formation of the beam) L, M, O, are partial differential coefficients of the same function of x and y, which we may call F: so that

$$\text{L} = \frac{d^2\text{F}}{dy^2}, \quad \text{M} = \frac{d^2\text{F}}{dx dy}, \quad \text{O} = \frac{d^2\text{F}}{dx^2}.$$

Substituting these, the equations become

$$\int . d\left(\frac{d\mathrm{F}}{dy}\right) = 0, \qquad \int . d\left(\frac{d\mathrm{F}}{dx}\right) - \mathrm{R} = 0,$$

$$\int . d\left(y\frac{d\mathrm{F}}{dy} + x\frac{d\mathrm{F}}{dx} - \mathrm{F}\right) - \mathrm{R}h = 0.$$

Considerations of a somewhat detailed character, depending partly on the relation assumed to exist between tension-force and material extension, are necessary to show the form which must be assumed for F in the various cases to be examined. The conditions to be secured are—that the horizontal part of the thrust, &c., shall be the same as that given by ordinary theories, on the relation just mentioned; and that the equations above shall be satisfied. After due application of these in the following five cases, these forms are found for F.

Case 1. A beam of length r and depth s projecting from a wall;

$$\mathrm{F} = \frac{6}{s^2} \cdot (r - x)^2 . \left(\frac{sy^2}{4} - \frac{y^3}{6}\right).$$

Case 2. A beam of length $2r$ and depth s supported at both ends;

$$\mathrm{F} = \frac{6}{s^2} . (x^2 - 2rx) . \left(\frac{sy^2}{4} - \frac{y^3}{6}\right)$$

Case 3. A beam like the last, carrying a weight W at the distance a from one end.

In this case the function is discontinuous; its forms are—

From $x = 0$ to $x = a$,

$$\mathrm{F} = \frac{6}{s^2}\left\{\left(2r + \mathrm{W}\frac{2r - a}{rs}\right)x - x^2\right\} . \left(\frac{y^3}{6} - \frac{sy^2}{4}\right).$$

From $x = a$ to $x = 2r$,

$$\mathrm{F} = \frac{6}{s^2}\left\{\frac{2\mathrm{W}a}{s} + \left(2r - \mathrm{W}\frac{a}{rs}\right)x - x^2\right\} . \left(\frac{y^3}{6} - \frac{sy^2}{4}\right).$$

(Of this case, two instances are given in the curves below.)

Case 4. A beam like that in Case 2, with a straining momentum applied at each end, as in the middle tubes of the Britannia bridge;

$$\mathrm{F} = \frac{6x^2 - 12rx + 3r^2}{s^2}\left(\frac{sy^2}{4} - \frac{y^3}{6}\right).$$

Case 5. A beam like that in Case 2, with a straining momentum applied at one end only, as in the exterior tubes of the Britannia bridge;

$$\text{F} = \frac{(6x-12r)\left(x-\frac{r}{4}\right)}{s^2}\left(\frac{sy^2}{4}-\frac{y^3}{6}\right).$$

By forming the differential coefficients of F symbolically, L, M, and Q $(=y-\text{O})$ are obtained in a form which admits of numerical calculation for every value of x and y. And from these, B, C, and β are computed without difficulty.

In this way the values of B, C, and β have been found for every combination of the values $x = r \times 0{\cdot}1$, $x = r \times 0{\cdot}2$, $x = r \times 0{\cdot}3$, &c., with the values $y = s \times 0{\cdot}1$, $y = s \times 0{\cdot}2$, $y = s \times 0{\cdot}3$, &c. In Case 1, 121 points were thus treated: in each of the other cases the computations were made for 231 points.

In the following diagrams are given the curves representing the directions of pressure and tension through the beam, together with a few numerical values at the most critical points, for each of the cases to which allusion has been made.

CURVES REPRESENTING THE STRAINS IN BEAMS, UNDER DIFFERENT CIRCUMSTANCES.

The continuous curves indicate the direction of thrust or compression; the interrupted curves or chain lines indicate the direction of pull or tension.

The figures denote the measure of the strain; the sign + meaning compression, and — meaning tension. The unit of strain is the weight of material lamina whose length = depth of beam.

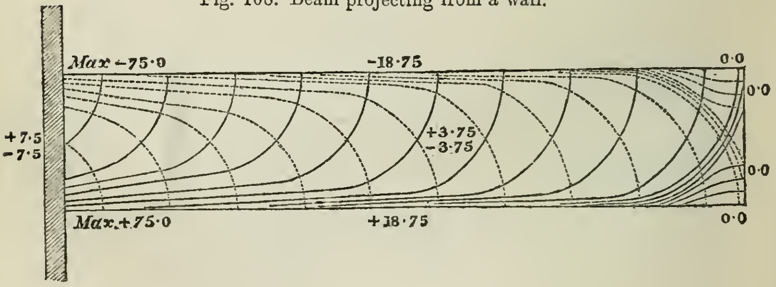


Fig. 108. Beam projecting from a wall.

Fig. 109. Beam supported at its ends.

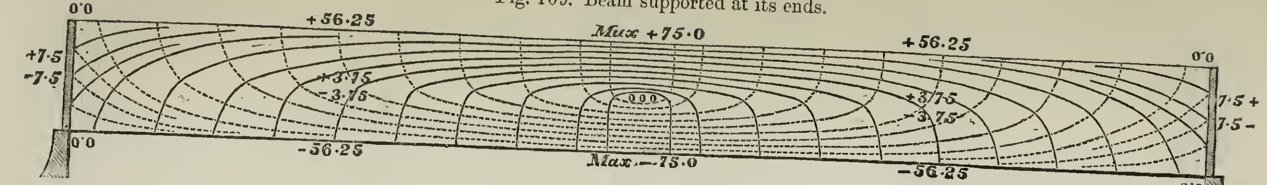


Fig. 110. Beam supported at its ends, and carrying, at the middle of its length, a weight equal to half the weight of the beam.

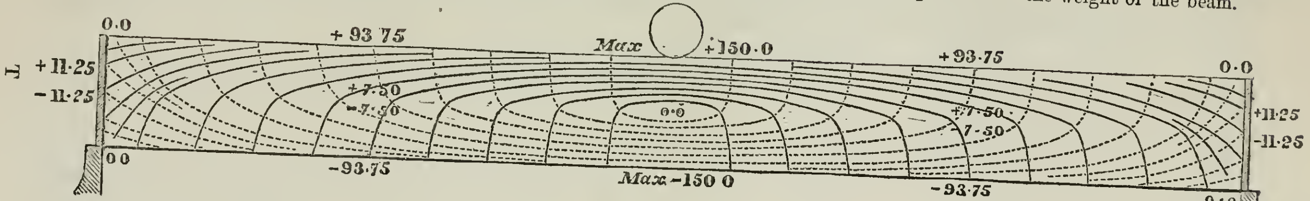


Fig. 111. Beam supported at both ends, and carrying, at the middle of a half length, a weight equal to half the weight of the beam.

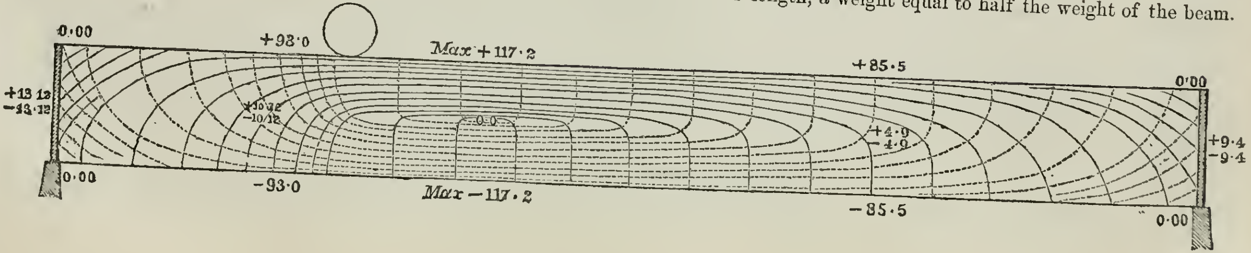


Fig. 112. Beam supported at both ends, and in which a strain (of the nature of a moment or couple) is impressed on each end, as in the interior tubes of the Britannia Bridge.

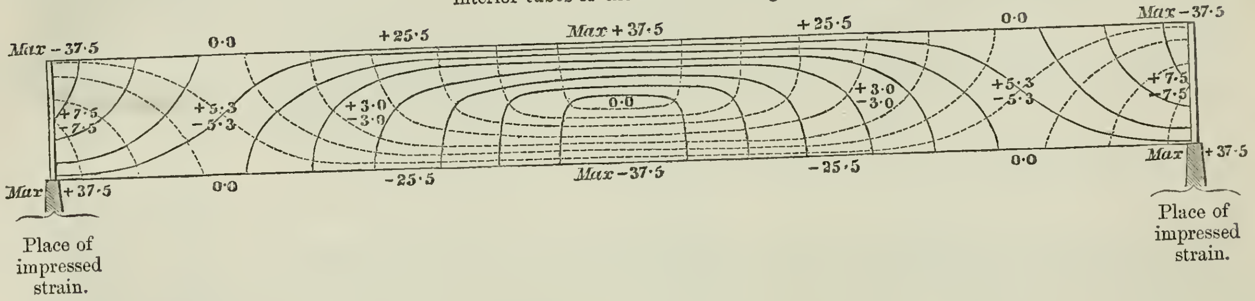
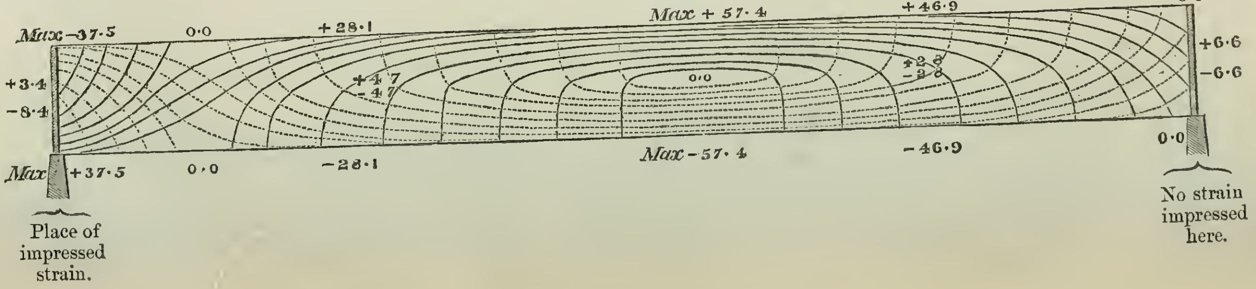


Fig. 113. Beam supported at both ends, and in which a strain (of the nature of a moment or couple) is impressed on one end, as in the exterior tubes of the Britannia Bridge.



THE PROPOSED TUBULAR AND TUBULAR GIRDER BRIDGES, FOR CROSSING THE RHINE AT COLOGNE.

During the progress of the construction of the Britannia and Conway Tubular bridges, and shortly after the completion of the latter, in October, 1849, I was invited by his Excellency the Prussian Minister, Chevalier Bunsen, to visit Berlin and the Rhenish Provinces, for the purpose of conferring with the authorities on the expediency of erecting a tubular bridge for carrying the railway and general traffic across the Rhine at Cologne.

Some time previous to that visit a chain suspension bridge from the designs of the Government engineer had received the sanction of the Government, and preparations were being made to carry it into effect.

It did not occur to the author of these designs that the flexibility of a bridge of this character would render it unsuited to the support of railway traffic, and to remedy this serious defect it was intended to split the trains into sections, and after raising them by machinery to the required level of the bridge, to drag them piecemeal by means of horses from one side of the river to the other. In this way the passage was to be effected, and certainly a more complicated and unsatisfactory plan could scarcely have been devised; in fact, one better calculated to create delay and inconvenience could hardly have been adopted. This very objectionable method of connecting the right and left banks of the Rhine,—this cutting in twain the main artery of communication between eastern and western Prussia, at a point above all others the most important to the public,—received the sanction of the Minister of Public Works, and to this was appended the signature of his Majesty the King. In this position of affairs, being called upon to offer new designs, I had to contend, on the one hand, with the preconceived opinions of the engineers and the plans to which they were committed, and on the other to prove to the Government the necessity of commencing an entirely new system of operations in order to meet all the requirements of the railway and general traffic.

This was a task which required the utmost prudence and circumspection, and that more particularly when were considered: first, the total inefficiency of the proposed plans; second, the opposition likely to be created by the authors of the chain bridge to any suggestion that might be made for a new and more perfect structure; lastly, the necessity of proving by direct experiment and the announcement of undeniable facts, that works of a similar character and much greater magnitude had been executed in England; and hence that there was no difficulty in the construction of a bridge across the Rhine, calculated not only to meet all the requirements of a large passenger and carriage traffic, but capable of supporting two lines of rails and the heaviest railway trains at full speed.

In the performance of those duties, there were no serious difficulties to encounter if supported by the authority of the Government and the Minister of Public Works, but appearances soon indicated opposition in that quarter. Obstructions to the navigation, the separate interests of the municipality of Cologne, and other objections, were brought forward to procrastinate and ultimately to prevent the introduction of another and more permanent structure.

It was fully admitted by all parties, and among them by the Minister of Public Works, that the tubular system met all the requirements of the case, and that it would effect within itself not only a safe and commodious carriage-way admirably adapted for the local traffic between the two towns of Cologne and Deutz, but, what was of much greater importance, it would secure the unbroken continuity of the great railway thoroughfare, so as to unite, by an undivided chain of communication, the two extremes of the kingdom from Belgium to the Russian frontier. His Majesty the King, the learned Baron Von Humboldt, and other distinguished persons connected with the Administration, were fully alive to the importance of this undertaking; and although the suspension bridge had received the royal assent and the approval of the executive, it was at his Majesty's request laid aside until the Government had made itself acquainted with the tubular system and its applicability to bridges of great strength and rigidity and of wide span.

To satisfy the public mind, and to give time and opportunity for collecting information on the subject, it was deemed expedient to send a commission to England to investigate the subject and report upon the various bridges to which it had access, and more particularly on those of the tubular kind submitted for inspection. These measures were, however, so carefully concocted, that investigation was evidently not the object to be obtained, but to gain time, and, if possible, to defeat the whole scheme of the tubular system; and this became strikingly apparent when it was found, from an article in the *Times*, April 15th, 1850, that the tubular system, notwithstanding its success in the completion of the Britannia and Conway Bridges, had not found favour with the Bureau of Public Works. The following extract from that article will fully explain the views of the authorities at Berlin at the time:—

'*The Bridge over the Rhine at Cologne.*—During the course of autumn, 1849, Mr. Fairbairn, of Manchester, was induced by representations made to him through a high official functionary, to propose to the Prussian Government a plan for an iron bridge across the Rhine at Cologne, on the tubular principle. This plan met with the entire approbation of the scientific world at Berlin, was sanctioned by the King, and was all but adopted by the Prussian Cabinet. It happened, however, that simultaneously with the proposal of Mr. Fairbairn, one Oberbaurath Lentze had become convinced that a suspension bridge was the true means of communication across the Rhine. He had arrived at this conclusion after years of patient investigation, and so far worthy of all praise, though it somewhat unfortunately chanced that his discovery was some thirty years too late, so that his labours, which would have been at the height of science in 1820, only served to illustrate a job in 1850. Here, in England, we have some little notion of the nature of jobs, but we question whether any more colossal in this kind has ever been perpetrated in our palmiest days of corruption than the attempt of our worthy friend Herr Van der Heydt, in whose paper, if we are not mistaken, the celebrated figment of the payment of 6,000*l.* to the editors of the *Times* by the Danish Government first appeared, to bolster up the scheme of M. Lentze and to throw cold water upon that of Mr. Fairbairn. What mattered it that Baron Humboldt, the Nestor of physical science, sided with Mr. Fairbairn, or that the King of Prussia in one of his happy moments had graciously extended his royal protection to the English engineer? Was not M. Van der Heydt, and the whole army of Prussian bureaucracy, whose name is Legion, arrayed on the other side? Still the English scheme must be burked officially; it was to be smothered in due form with the Cushion of bureaucracy; so a commission was appointed to inquire into the English tubular bridges; and of whom do our readers suppose that it consisted? Why, of Herr Oberbaurath Lentze himself and another person, who after due deliberation set off for England on their scientific mission. Why need we detail at length the wanderings of these duumviri of bureaucracy?—how they landed in England—how they were received with marked courtesy by Mr. Fairbairn—how they saw the Conway Bridge, the Britannia Bridge, and other structures of minor dimensions in Lancashire? Suffice it to say, that they were quite blind to the merits of tubular bridges, and made their report dead against tubular bridges and strongly in favour of suspension bridges—a report which was adopted by the Government, that is to say, by Herr Van der Heydt, who forthwith issued his famous notice calling upon the engineers of the whole world to compete for the honour of contributing to the glorification of Herr Oberbaurath Lentze, whose plans have been long since in the bureau of M. Van der Heydt, and in all probability will be ultimately carried out.'

Such was the manœuvring and such the result of the negotiations for the erection of a bridge on the tubular principle over the Rhine. Eight years have elapsed since the project was first brought under the notice of the Prussian Government. Several years were taken for its consideration, and a further period of two years will still be required before the structure arrives at completion.

It is not my place to say whether the present structure or its site be superior or inferior to the one I had the honour to propose; that is a question for others to determine. But, taking into account the magnitude of the structure, and the accommodation it was calculated to afford for a continuous railway and general traffic, I have considered the history of its origin and the general character of the work of such importance as to entitle it to a separate notice, descriptive of the outline of a project which may yet be useful to the public, and of some interest in elucidating the means of easy and convenient communication between countries divided by wide and rapid rivers.*

Whilst the importance of a firm and solid rail and roadway bridge over wide rivers such as the Rhine cannot be too highly estimated, either as a work of art or as forming the connecting link between the extreme parts of an empire, yet its erection was entitled to serious consideration as a work of considerable difficulty. To construct it of wide span so as neither to impede the navigation nor contract the waterway, to allow sufficiently large openings for the passage of ice in winter and of the huge rafts of timber which are floated down the river in summer, would have been a work of no ordinary description, and one surrounded by difficulties which would have required the utmost care and attention to overcome.

These requirements would, in my opinion, have been best attained by a tubular or tubular girder bridge, and accordingly the following projects were submitted to the authorities at Berlin as the most eligible for the objects contemplated. Two designs were given: one, for a bridge of four spans on the tubular girder system, as the least expensive; the other, of two spans, through the tubes of which the up and down trains should pass. In both designs, a carriage-way for street and road traffic was formed between the tubes, and capacious galleries, resting on cantilevers, were projected from the tubes on each side for the convenience of pedestrians.

These bridges, from their great strength and rigidity, unite a permanence equal to that of stone with a width of span rivalling that of the suspension bridge. It may indeed be said, that they do not possess the strength and simple beauty of the arch, nor the aërial lightness and harmony of proportion of the catenary, but they contain the elements of both, and are admirably fitted for the work they have to perform.

On referring to the plates it will be seen that the projected bridges unite the following peculiar advantages:

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* Some further information on this subject will be found in a letter to Baron Von Humboldt given in Appendix V.

1st. They allow the passage of trains at full speed, at all times, and in every state of the river.

2nd. They provide a broad carriage-way running parallel with the railway but separated from it, in one case by the centre girder, in the other by the sides of the large tubes.

3rd. They have two spacious galleries, each 10 feet wide, separated from the railway and roadway by the tubes or main girders, and affording a promenade in full view of the river, its shipping, the city of Cologne, and the distant mountains.

Lastly, they have a pair of swivel bridges, one on each side of the river, for the passage of the largest vessels, leaving only the smaller craft to pass under the bridge.*

In the approaches to the bridge, carriages would enter from the Cologne side by the street which intersects at the east end of the Cathedral (plate 3), and, having crossed the bridge, would by an easy curve and descent enter the town of Deutz by the street marked *Fahrweg* on the plan.

The height of the bridge necessary for allowing the navigation of the river would permit the Aix-la-Chapelle line to be continued on arches, at its present level, from the existing temporary station along the Quay (*Bahnhof der Rheinischen Eisenbahn*) to the general station, shown on the plan, opposite the Cathedral. Hence the station would be on the same level as the roadway of the bridge, and under the arches on which it is supported would be the dépôt for goods and merchandise.

On the opposite side at Deutz, the railway would descend from the level of the bridge to that of the Deutz and Minden line, at a point where the gradient of 1 in 100 or 1 in 120 would terminate.

These bridges are of greater strength than any of equal span hitherto constructed, and are calculated to resist the vibrations caused by a train running at its greatest speed or a park of artillery at full gallop.

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* It was strongly urged upon the Government at Berlin to abandon the drawbridges, but this was decidedly objected to at the time. I have since learned, that in the bridge now erecting large vessels must strike their masts, as no provision will be made for their passage through the bridge.

Explanation of the plates. Plan, plate 3, showing site of Bridge. The plan shows the Cathedral, the lower portion of the town of Cologne, part of the town of Deutz, and the existing stations of the Rhenish and Cöln and Minden Railways. The bridge, as will be seen, crosses the Rhine directly opposite the eastern windows of the Cathedral, and terminates on the other side near the station of the Cöln and Minden Railway. Carriages and other traffic enter the bridge below the Cathedral, and having crossed the bridge descend by an incline into the town of Deutz. Foot-passengers may ascend the inclined road, or the steps at the land-piers of the bridge, and pass the bridge on either side.

Plate IV. Elevation and plan of a tubular girder bridge. The bridge consists of four spans, the two middle spans being each 326 feet from centre to centre of the piers, and the two end ones each 244 feet 6 inches span to meet the drawbridges. The drawbridges revolve upon rollers, moving on iron platforms embedded in the masonry, and support the roadway above. This platform would be moved by machinery through the quadrature of a circle to permit the passage of large vessels, and by the same process moved back till it met a stop which rendered it immovable till it should be again opened. These revolving platforms are each 203 feet 9 inches long, and open an uninterrupted passage 70 feet wide for the navigation of the river by large vessels.

Fig. 114 is a transverse section of the bridge at the centre. It exhibits the position and proportions of the girders, the width of the railway, roadway, and foothpaths, and the arrangement of cross-beams, cantilevers, &c., for supporting each.

The footpaths extend the whole length of the bridge, and would be protected by a strong and handsome balustrade.

On the cross-beams supporting the roadway, cast-iron plates, four feet square, would be laid with projecting ribs on their upper surface for receiving a pavement of wooden blocks; and it was intended to fill up the spaces above the dividing ribs with a composition of hard pulverised limestone or granite, and pitch, rendering the whole compact and impervious to wet.

The permanent way over the railway and footpath would be laid with 3-inch planking, bolted to the cross-beams and cantilevers; that over the railway being covered with sand or gravel to a depth of about 2 inches to prevent danger from the sparks or cinders of the engine.

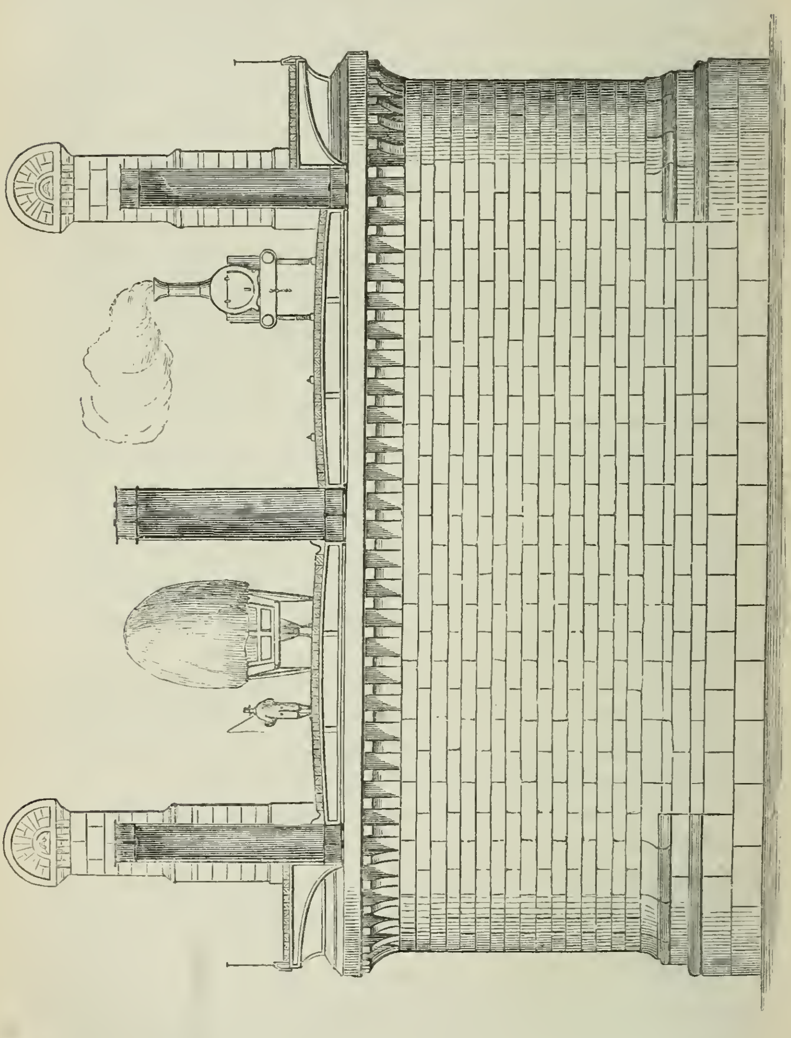


Fig. 114.

The computed strength of the bridge is equivalent to a strain of 12 tons per lineal foot for the railway, and 8 tons per lineal foot for the carriage-way and footpaths; making the bearing powers of one foot across every part of the bridge equal to 20 tons. This is equivalent to 21,000 tons, *minus* 5,000 tons, the estimated weight of the bridge, or 16,000 tons, equally distributed over its surface. The greatest load which could ever be brought upon the bridge would be to cover the railway with loaded waggons and locomotives, and the carriage-way with a drove of cattle; or both carriage-way and footpaths with a dense crowd of people. This would be equivalent to about 2,700 tons, or $2\frac{1}{2}$ tons per lineal foot. So that the ratio of the breaking load to the greatest weight which could come upon the bridge would therefore be as 16,000 : 2,700, or about as 6 : 1.

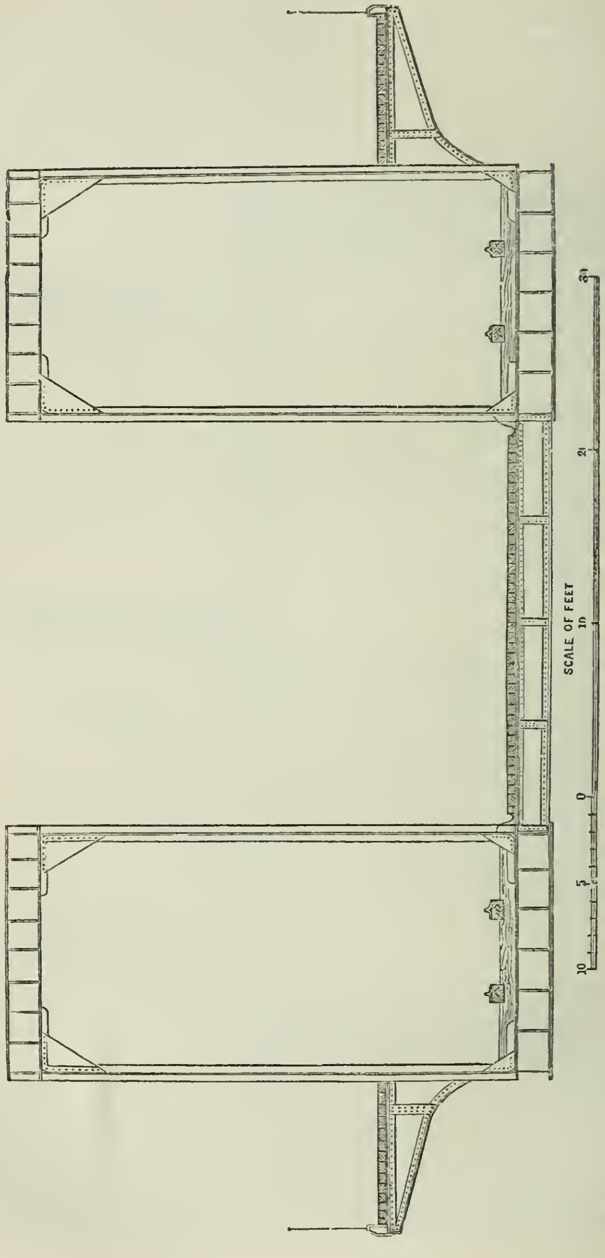
The bearing powers of the bridge are therefore greatly in excess of the load, a circumstance of vital importance in affording sufficient security against the vibration of a large and continuous traffic.

The estimated cost of the iron superstructure of this tubular girder bridge is as follows:—

	£	*s.*	*d.*
For the large tubular girders 526 wrought-iron cross-beams, 526 cantilevers for supporting footpaths; 21,000 square feet of cast-iron base plates, expansion and contraction rollers and apparatus; delivered free on board, at an English port . .	110,000	0	0
Balustrades; packing, &c., for footpaths. Two revolving drawbridges, including the requisite machinery for working the same	36,000	0	0
Total	£146,000	0	0

In the above approximate estimate of ironwork, it is assumed that the Government will construct the piers, cofferdams, &c., and find all the necessary material for scaffolding and forming the roadway. To this would have to be added the freights, carriage, re-erection, &c., all of which might amount to 200,000*l.*, for the work in its finished state; and taking into account the massive piers, with foundations for the swing-bridges, scaffolding, arches, and approaches on each side, double that

Fig. 115.



sum, or 400,000*l.* sterling, would be required to complete the bridge.

The tubular bridge, of which Plate VI. shows a plan and elevation, and fig. 115 a cross section through the centre. This bridge would cross the Rhine at the same point as the last, and the approaches to it would be nearly the same. It consists of two large wrought-iron tubes running parallel with each other, through which the railway trains would pass. Each of these tubes is continuous, and rests on three piers at distances of 570 feet 6 inches from each other. Between the tubes is the carriage-way, which is shown 24 feet wide but might be made 30 feet if required. Along the exterior of each tube are attached the galleries or footpaths supported by riveted cantilevers.

The lower side of the tubes, calculated to resist tension, would be perfectly horizontal, and, like the top, which has to resist compression, would be made cellular. The top would be a parabolic curve, that the girder might be deeper in the centre than at the ends. At the termination of the large tubes would be swivel bridges, as in the former case; these when opened would leave a space of 70 feet on each side of the river for the passage of large vessels.

The peculiar characteristic of this structure is the enormous span on each side of the central pier, exceeding that of the Britannia Bridge by about 100 feet. The great advantage of this is the free passage it affords for the flood waters of the Rhine, the large quantities of ice, and the timber rafts which float down the stream.

Another striking feature of this structure is its immense strength and the resistance it offers to the impact of heavy railway trains or the oscillation caused by violent winds.

In this bridge the strength is much greater than in the last, being increased in the ratio of the enlargement of the span. Ten tons per lineal foot is the estimated strength of each tube, which is equivalent to 5,500 tons placed in the centre of each span. The resisting powers of this bridge, after deducting its weight, would be about the same as in the former case; that is to say, 16,000 tons equally distributed over the surface; or the ratio of the breaking-weight to heaviest load would be as 6 : 1.

The estimated cost of the bridge would be as follows:—

	£	s.	d.
The tubes, cross-beams, cantilevers, &c. are estimated at	170,000	0	0
Two drawbridges and the requisite machinery, cast-iron frames, expansion rollers, balustrades, footpaths, painting, &c.	50,000	0	0
Making a total of	£220,000	0	0

for the iron work delivered free at an English port. To this must be added all the expenses of freight, carriage, reconstruction, masonry, scaffolding, &c., which cannot be estimated at less than 240,000*l.* or 250,000*l.*, making a gross total of 470,000*l.* to complete a railway and roadway tubular bridge of two spans over the Rhine.

APPENDIX.

No. I.

REMARKS RELATIVE TO WROUGHT-IRON BEAMS.

CONSIDERABLE doubts have been entertained respecting the efficiency of wrought-iron beams for the support of arches forming the floors of fire-proof buildings. This description of beam has lately been used for various purposes, such as those for horizontal bridges and the support of wooden floors in buildings; but their application has been confined within exceedingly narrow limits; and until they were first applied experimentally at Wolverhampton, and subsequently, on a larger scale, at Saltaire, for the support of brick arches, they were previously considered, from their ductility and liability to lateral flexure, to be unfit for such a purpose. These examples have, however, proved this opinion to be incorrect, as we have upwards of 100 brick arches supported on wrought-iron beams, on the tubular-girder bridges which cross the river Aire at Saltaire, with a degree of rigidity fully equal to that of cast-iron. The same may be said of the experimental arches in Messrs. John and James Norton's corn-mill, where the arches are supported with equal solidity, and with more security, than those which spring from cast-iron beams.

I have deemed it necessary to give these facts, in order to prove the superior efficiency of the wrought-iron beam; their comparative lightness, being only *one-third* of those composed of cast-iron; and their greater security from fracture, whether arising from a dead weight or the force of impact.

No. II.

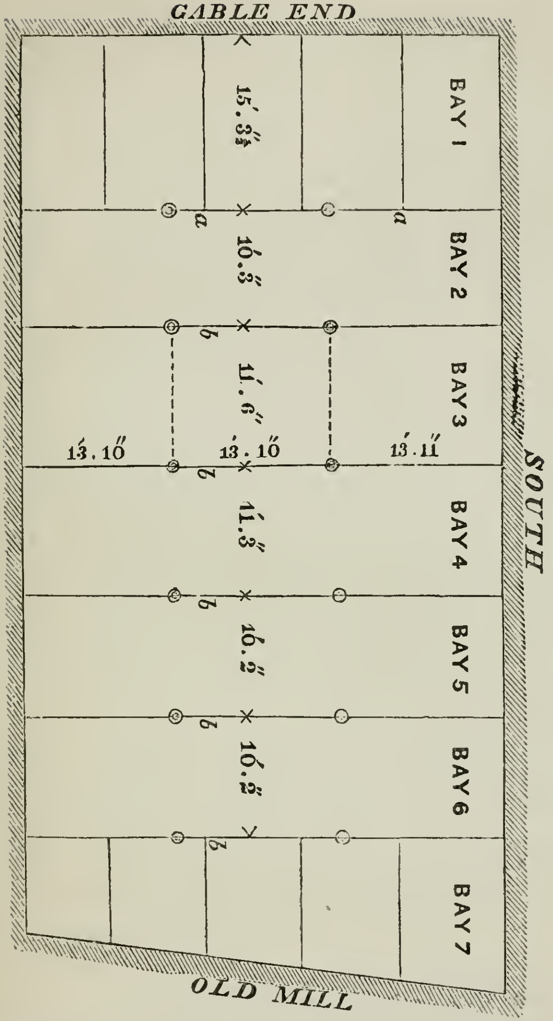
REPORT ON THE CAUSES OF THE FALL OF THE COTTON-MILL AT OLDHAM, IN OCTOBER, 1844.

WE carefully examined the building, and having noted every particular relative to the walls, foundations, iron beams, columns, and their fractures, are of opinion that the accident has arisen from one of two causes: namely, from the falling of the arches in the first instance; or, what is more probable, from the breakage of one of the large beams supporting the transverse and longitudinal arches at the extreme gable of the mill.

From the evidence already adduced it appears that one of the arches in the top room (the fourth from the old mill) was observed to sink some days previously to the accident. This arch, which had sunk about four inches, was considered unsafe, and the necessary preparations were immediately made for refixing the centres with a view to its renewal. During the rebuilding of the arch (when about one-third of it was completed, the middle having been removed, and the other parts remaining) the building, at this critical period, gave way, and, as stated by one of the witnesses, the beam broke short by the column, and the whole came down with a crash. Now, in this view of the case, assuming the evidence to be correct, it is obvious that the beam must have been broken from the lateral strain of the arches, and not from the weight acting vertically (as assumed) upon the beams which remained. In confirmation of this opinion, it may be observed, that the middle beams were unprotected from the lateral thrust, unless we except an imperfect wooden stay, which, from its soft and fibrous nature, would easily split or crush by the force of the edge of a flanch of only one inch thick pressing upon it. Hence it follows, that the thrust of two wide and flat arches would be quite sufficient to fracture the beam, and thus loosen or destroy the abutments on each side. The beam being ruptured, it is easy to conceive the result which must inevitably follow. From the breakage of this beam we may infer a serious and extensive accident; but, to our minds, it does not sufficiently clear up the full amount of injury sustained, nor does it account for the immense crash and total destruction of the building which took place. One of the middle beams, or any one single beam of the building, giving way, could not, in our opinion, have made the ruin so complete; and, having reason to suspect some other cause, we were induced to institute a still more minute and searching inquiry into the strength and proportions of other parts of the structure.

On a careful examination of the fractured beams, and more particularly of those which stretch transversely across the building, at a distance of 15 feet from the entrance-gable of the mill, we found a more convincing proof of the cause which led to this unfortunate occurrence. These beams carry the ends of four other beams, which extend longitudinally from the gable on which they rest, as shown in the following sketch.

Fig. 116.



From the above sketch, it will be evident that the beams *a*, *a* had to support a much greater weight than the beams *b*, *b*, *b*, &c.; consequently they required to be made of proportionately greater strength. They were made stronger; but unfortunately, from inadvertency, or rather from want of knowledge, they were strengthened in the wrong place; and instead of additional strength being given to the bottom flanch, which is always subjected to the greatest strain, it was given to the middle of the beam, where it was not required. It is well-known, or it ought to be known to every person giving instructions for the form and construction of iron beams, that the strength is nearly in proportion to the section of the bottom rib or flanch, other things being the same; for, according to Mr. Hodgkinson's experiments, a bottom flanch of double the size will give nearly double the strength.

These facts having been proved by direct experiment, it is important to all those concerned in the construction of fire-proof buildings, in which life and property are at stake, that the form of beams and the section of greatest strength should be thoroughly understood;* and we would beg to refer those unacquainted with the subject to Mr. Hodgkinson's paper on the strength of iron beams in the fifth volume, second series, of the Memoirs of the Literary and Philosophical Society of Manchester. In ordinary cases we should not have troubled the jury with these remarks; but in a case of such importance as the present, where the lives of so many persons have been sacrificed,† in consequence of defective knowledge and want of skill in the construction of buildings, we have considered it our duty thus publicly to direct attention to the subject, not only as regards the present but all future cases, and respectfully to urge upon the proprietors of mills, and of other buildings containing work-people, the necessity for a more secure and perfect system of building, and for a further development of the principles upon which fire-proof edifices are based. If this suggestion is properly received and acted upon, we have reason to believe that we shall not again have occasion to investigate occurrences of so lamentable and so distressing a nature.

We have already observed that the beams *a*, *a*, *a* in the preceding sketch were strengthened, not, however, in the bottom flanch, but in the middle part of the beam, where they were thickened, and where the augmentation of strength was almost of no use. Had the same quantity of metal been given to the lower flanch, these beams (the weakest in the building) would have carried nearly double the weight; and thus, by a proper and judicious distribution of the material, the building, as well as the lives of the people, would have been saved.

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* In order to show the importance of at least a knowledge of first principles when applied to practice, we have only to take a beam with a single flanch at the bottom—thus ⊤—and break it with the flanch downwards, with a weight say 1,000. Reverse a similar beam from the same model, and again break it with the flanch upwards—thus ⊥—and we shall find that it only requires 320 to 340 to break it. Most people are totally unacquainted with these facts; and the great majority suppose a beam to be as strong laid the one way as the other. Hence the anomalous position in which we are sometimes placed.

† Upwards of twenty persons were killed on the occasion.

These observations apply to all the other beams of the mill, which are also defective as respects their strength.* Those transverse beams sustaining the ends of the longitudinal beams were acted upon with a load of 13¾ tons.

Now, if we take the sections of these beams, and calculate the weights necessary to break them when laid upon the middle, it will be found that the breaking-weights for the beams *a*, *a*, *a*, and *b*, *b*, *b*, &c., fig. 116, will be nearly the same, or about 9¼ tons. This is the breaking-weight of an average quality of iron; and, allowing for the difference of metals, it could not be raised much above 10 or 10½ tons. The breaking-weight would, therefore, be about 10 tons when the beam is loaded in the middle, and 20 tons when the weight is equally distributed over the whole surface of the projecting flanch of the beam.

Having ascertained the bearing powers of the beams, we shall next compare their strength with the actual loads they were required to sustain; and in making that comparison, it must be borne in mind that the two beams *a*, *a*, fig. 116, next to the side wall, had their load unequally distributed, which reduced their bearing powers to 15 tons.

You will now perceive that, on the west side, the beam *a* was able to carry 10 tons; but the cross beams, on the east side, threw the whole weight upon the middle of the beam; and consequently, instead of the breaking-weight of the beam *a* being 20 tons (as it would have been if equally distributed) it was only 15 tons, the weight being distributed only on one side, the weight on the other side bearing entirely upon one point.† Now the load which these beams had to support was 13¾ tons; 8½ tons being supported on a single point on one side, and 5¼ tons distributed over the surface of the opposite flanch.

From this it will be seen that the actual load was to the breaking-weight as the numbers 13·75 to 15, or as 1 to 1·09, showing that the actual load was within a mere fraction (one-tenth) of the weight which would crush the beam. Such was the critical state in which this building was standing just previous to the fall.

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* In computing the weights on each of the beams, it was found that those supporting the arches of 10 feet 6 inches, and those of 11 feet span, had to support loads of 10 to 11 tons respectively, without machinery. These, when loaded with machinery in motion, gave a near approach to the breaking-weight, which, it will be observed, was only 19 to 20 tons.

† There is a wide difference between loading a beam on one point in or near the middle, and loading it along its whole length. In the latter case it would carry just double the weight; consequently, in beams supporting arches, we have an equally distributed load; so that a beam which would break with 10 tons applied to a single point in the middle, will sustain 20 tons, or double the weight, if distributed.

Viewing the subject in this light, and taking the above calculations as data, we are no longer at a loss in relation to the cause of the accident. Even supposing the arches to have stood, it will be obvious that so close an approximation of the actual load to the breaking-weight was extremely unsafe; and that, under such circumstances, no precautions could have prevented the rupture of the transverse beams *a*, *a*, *a*, fig. 116, whenever they happened to be subjected to the slightest impact, or to any vibratory motion tending to disturb the parts under strain, and eventually still further to lessen their already too-much diminished powers of resistance. It is clear that they must have gone at some time or other. I believe Mr. Bellhouse to be of the same opinion with myself, that this was the real cause of the accident,—that probably, from the vibratory action of the mill-gearing on being started, or from some other cause, the slightest shock would fracture either of the beams *a*, *a*, and it is easy to conceive how the others would then follow. It would not only carry the gable-end down, but it would loosen the arches on the same floor, and the whole would inevitably come down in the mass. It would be impossible to account for the entire destruction of the building unless the frame-work of one floor came down ; and one of these beams *a*, *a* giving way would account for such a catastrophe.

Irrespectively of the weakness of the iron beams, which we consider as the primary cause of the accident, we would beg to advert to the tie-rods, which, although sufficient in number and strength, are not judiciously placed as respects their position for resisting the strain of the arch, their maximum point of tension being at the bottom flanch of the beam : that being inconvenient, they should on no account be placed higher than the soffit of the arch ; and in this position they would perforate the neutral axis of the beam, and give sufficient security to the arch, without impairing the strength of the beam. Instead, however, of approaching this point, they were placed on the top of the beam, and 18 inches from the bottom flanch.*

As respects the arches, we found the versed sine or rise of the arch too low. On most occasions they are $1\frac{1}{4}$ inch to the foot; but in order to insure perfect security we should advise, in all future buildings of this description, that the rise be $1\frac{1}{2}$ inch to every foot of span. In the arch which first gave way, the rise was only a small fraction above an inch, having a rise of only 12 inches in a span of 11 feet 6 inches.

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* In every description of arch supported by iron beams, it is essential to have the tie-rods as low as possible; it is in most cases inconvenient to have them in the line of the chord of the arch, or the bottom flanch of the beam; but in every instance they should never be higher than the soffit of the arch, and in this position they would perforate the neutral axis, as stated above.

On viewing the columns, several imperfections were observed in the variable thickness of the metal ; but in other respects the pillars were satisfactory, and presented no features of weakness indicating danger from those parts: one inch more in diameter, with the same weight of metal, would, however, have given greater strength and greater security.

We cannot close this report without adverting to the anxious solicitude of Messrs. Radcliffe, and the strong desire evinced by them to have every part of the structure upon the best and strongest principle ; and we should imperfectly discharge our duty on this occasion, if we neglected to bear testimony to the superior strength of all parts of the building, except those which we have thus described, and on which they could not be expected to be able to form an opinion. We cannot expect that gentlemen who are not acquainted with the principles of construction should have an adequate knowledge of all the proportions and other conditions which are requisite in such a building. In conclusion, we have great pleasure in stating that it appears to us that no mere pecuniary considerations were present to the minds of Messrs. Radcliffe in the construction of these mills.

(Signed) William Fairbairn,
David Bellhouse.

No. III.

On some Defects in the Principle and Construction of Fire-proof Buildings.

By William Fairbairn, M. Inst. C.E.

Read before the Members of the Institution of Civil Engineers, April 20, 1847:
Sir John Rennie, President, *in the Chair.*

The falling of a portion of Messrs. J. and J. L. Gray's cotton-mill, at Manchester, is a striking example of the dangerous consequences which may arise from the use of cast-iron beams of large span without adequate pillars or support. Except in cases of absolute necessity, the superincumbent mass of brick arches should never be supported by suspended girders. When pillars cannot be introduced, the size and strength of the girders become a question of importance. Under such circumstances, they should never be loaded beyond one-third of their breaking-weight. In the present instance, two columns might have been introduced between the boilers without detriment to any part of the structure. After having carefully examined the walls, iron beams, and other parts of the fallen building, and noted all the circumstances in connection with the accident, I can have no doubt as to the cause which led to the fracture of the beam, and to the consequences which ensued from it.

In order to arrive at a clear conception of the causes of this failure, it will be necessary for me to offer a few preliminary observations, which will not be considered as uselessly bestowed, on a subject of such vital importance as the security of buildings, on which depend not only the fortunes of the proprietors, but, what is of far greater moment, the lives of the numerous labourers engaged in the manufactory.

A crystalline metallic body, like cast-iron, when used for the floors of fire-proof buildings, bridges, &c., to support heavy weights, is an extremely treacherous material. It should be used with great caution, and only under the direction of competent persons thoroughly acquainted with all its physical and other properties, as well as its powers of resistance under different kinds of strain. In order to ensure safety, and to attain success in this respect, the following qualifications are necessary in the person who undertakes the construction of buildings of this kind :—

First, A knowledge of the properties and application of the material, when subjected to three distinct species of strain, namely, torsion, compression, and separation, or tearing the parts asunder.

Secondly, An exact knowledge of the proportion of the parts of a beam, so as to have the forces of extension and compression duly balanced when the beam is about to undergo rupture from a transverse strain.

Thirdly, A knowledge of the laws which govern the expansion and contraction of metals, in order to ensure sound castings, and an equal degree of tension during the process of cooling.

These remarks will be found applicable to all cases where cast and malleable iron are used in buildings ; and they are therefore introduced previously to the discussion of the question more immediately under consideration.

The building (longitudinal and transverse sections of which, as it stood before the accident, are shown in figs. 117 and 118) was about 40 feet long, and 31 feet 8 inches wide ; it consisted of two storeys, the lower floor containing three boilers, and the upper storey the machinery at which the work-people were employed. Immediately over the room last mentioned, and serving for the roof, there was a water-cistern, formed of asphalte, which covered the whole of the top of the building, an extent of about 1,270 square feet, and was intended to contain about

1 foot 2 inches depth of water. The floor over the boiler-room was composed of three large cast-iron beams, of 31 feet 8 inches in span, each 2 feet 3 inches deep at the centre, and 1 foot $10\frac{1}{2}$ inches at each end, trussed with wrought-iron bars $2\frac{1}{2}$ inches by 1 inch high, extending from wall to wall, without pillars or other supports. Between these beams brick arches were turned, which, being levelled and filled up in the groins in the usual manner, formed the floor for carrying the machinery in the upper storey, as exhibited in the annexed sections, figs. 101 and 102.

Fig. 117.

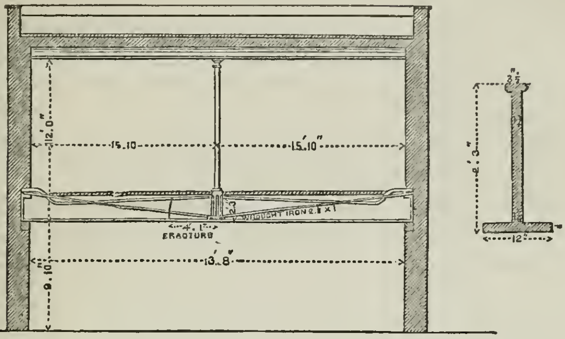
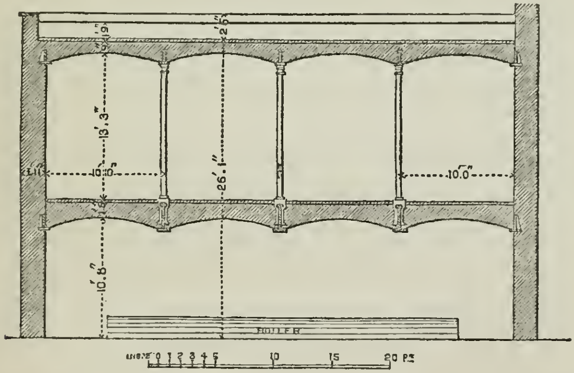


Fig. 118.



These three lower beams, with the walls, were the only supports for the iron columns, beams, and arches of the upper floor, including the cistern, which formed the roof. It will be seen, therefore, that they had not only to bear the weight of the brick arches and machinery of the first floor, but, in addition, half the weight of the iron beams, columns, and brick arches of the roof, as well as the water contained in the tank. Assuming this cistern to have been filled with water to a depth of only 18 inches (the height of the overflow-pipe), it will be found that these lower beams were totally inadequate to support the load thus resting upon them.

Previously, however, to entering upon the calculations, it is necessary to refer to the enlarged section of the girder (fig. 117), taken through the middle, where the depth of the beam was 27 inches, the area of the bottom flanch $19\frac{1}{2}$ square inches, and the area of the top flanch 3·6 square inches, the distance between the supports being, as before stated, 31 feet 8 inches. Now the breaking-weight of this beam, supposing it to be cast from metal of ordinary quality, would be about 36 tons. This is exclusive of the truss-rods, which, if properly applied to a well-proportioned beam, should increase its ultimate powers of resistance from 36 tons to 50 tons; but unfortunately these truss-rods were of little use, as the beam evidently broke from the crushing or lateral force on the upper side, which, from the nature of the fracture, was the first to give way, and consequently rendered the truss altogether inoperative. On comparing the strength of the broken beam with the load it had to sustain, it will be found, that instead of breaking when it did, it should have broken on the first trial, when the cistern was filled with water to a much greater depth.

In computing the weight of the floors, and of the water in the cistern, &c., the whole is reduced to a weight acting upon the middle of the beam; and as cast-iron beams will carry just double the weight when it is equally distributed over the surface, the whole weight acting in the middle is reduced to one-half the entire load spread uniformly over the top of the beam. The case is, therefore, nearly as follows:—

The weight of the brick arches on the beam which broke $= 20·5$ tons; the weight of the machinery on the same $= 10$ tons; and therefore the sum of these pressures acting in the middle of the beam $= \frac{20·5 + 10}{2} = 15·25$ tons. Again, the weight of the arches and of the iron beams of the roof $= 23·5$ tons; the weight of the water (18 inches deep) bearing on the beam $= 14$ tons; and therefore the sum of these two latter pressures acting on the middle of the beam $= \frac{23·5 + 14}{2} = 18·75$ tons, the total weight of the arched roof acting upon the middle of the beam. Hence the total load will be $15·25 + 18·75 = 34$ tons, the actual weight, when reduced to a force resting upon the middle of the beam. Now the breaking-weight of the beam is 36 tons; hence the ratio of the load to the breaking-weight is as 34 to 36, or as 1 to 1·06, showing very close approximation of the load to the breaking-weight.

It may be asked, how it was possible for beams so critically situated to sustain the excess of weight, when the cistern was filled with water to a much greater depth than 18 inches, and when each of the large beams would have to sustain a load of 41 tons in the middle, the breaking-weight being only 36 tons. In reply, it must be stated, that the metal in the beams appeared to be sound and good, and that on the first trial they had not been subjected to vibratory action, nor to the process of loading and unloading, by the alternate filling and discharging of the cistern, which they had subsequently to undergo, and to which may be attributed in a great degree the ultimate failure.

In the construction of the large girders, the great weight which they would have to sustain was evidently foreseen, as double truss-rods were introduced; but two important facts appear to have been lost sight of, namely, the weakness of the top rib, and the nature of the strain, where the whole weight of the top arches, &c., acted on one point, at the middle of the beam. The truss-rods were each $2\frac{1}{2}$ inches wide, and 1 inch thick; but they were reduced at the ends, by the holes for receiving the pins, which passed through the beams, and connected them together in the form of a link. Supposing the top rib of the girder to have been sufficiently strong, and the truss-rods equal in strength to their sectional area of $2\frac{1}{2}$ inches, the bearing powers of the beam would then have been increased (taking the tensile strain at 24 tons per square inch) to nearly 67 tons. But the misfortune was, that the top flanch was too weak to resist the action of the truss, which, having a tendency to crush the top, already deficient in the quantity of material, would rather diminish than increase its bearing powers. It is therefore evident that the three large girders, although not of the best form, were nevertheless a near approximation to it, when acting without trusses; but with those auxiliaries they were decidedly disproportionate, and more particularly defective in the top ribs, which rendered them exceedingly precarious, and decidedly unfit for supporting the load placed on them.

In closing these remarks it may be observed, that on several occasions the author has felt it an imperative duty to be very candid when called upon to investigate the causes of accidents wherein the lives of the public and the security of property were involved; and although still desirous of rendering assistance to the utmost of his ability, he is nevertheless unwilling to perform the onerous and invidious duty which, to a greater or less degree, may implicate the professional reputation of gentlemen of superior knowledge and acquirements.

In this instance, as in other investigations of the kind, he has not hesitated to declare the exact state of the case; but for the reasons above stated, he approached the present inquiry with some degree of reluctance; and if, in the course of the investigation, he has been under the necessity of condemning the principle as well as the practice upon which the building in question has been constructed, he feels convinced that his motives will be appreciated. In dealing with matters so intimately connected with public safety, the interests of companies and individuals are best served by a careful analysis, as well of the materials employed in the structure as of the principles of construction adopted in all its parts. It is a matter of considerable importance that engineers and architects should be thoroughly conversant with the strength and other properties of the material which they are employing; and none requires greater attention and consideration than cast-iron, to ensure confidence and security to all concerned. It is not a knowledge of form and construction only that is necessary, but also of the nature of tension and compression, as well as of the laws which govern the resistance under strain.*

No. IV.

The Box and the Plate Beam.

The difference in strength between the box and the plate beam, as referred to at p. 78, does not arise from any want of proportion in the top and bottom sections of either beam, but from the position of the material, which in that of the box form offers greatly superior powers of resistance to lateral flexure. Other means have been adopted for the purpose of increasing the lateral strength of the plate-beam; and that is, by screwing pieces of timber on each side of the plate, and thus, by increased stiffness, to render it less liable to warp under strain. This form, denominated the sandwich-beam, has been in some situations extensively used; it is composed of a plate, without angle-iron flanches, between two pieces of timber, depending probably more upon these adjuncts than any amount of strength likely to be obtained from the centre plate, which in such a position, and in such form, could not be considered as a beam. Having doubts as to the powers of resistance of this construction, I submitted it to the test of experiment, of which the following are the results.

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* For further particulars see the drawings and descriptions, &c., in the Transactions of the Institution of Civil Engineers.

The beam was 22 feet 6 inches long, 12 inches deep, and 12½ inches thick, with a solid plate the same length as the beam, also 12 inches deep, and ⅜ths of an inch thick. The side timbers were composed of good Baltic fir, and bolted on each side of the plate with 1-inch bolts. The beam thus supported was laid upon supports 22 feet asunder; and having prepared the apparatus, the weights were laid on as follows:—

Number of Experiment.	Weight in tons.		Deflection in inches.	Deflection when load was removed.	Remarks.
	Tons.	cwt.			
1	1	12	·25		
2	4	0	·50		
3	5	0	·75		The last load (18 tons) was left on the beam for 16 hours, and then removed, when the beam was found to be crippled to the extent of a permanent set of 1·30 inches.
4	8	0	1·00		
5	10	0	1·30		
6	12	0	1·50		
7	14	0	2·00		
8	15	0	2·25		
9	16	0	2·50		
10	17	0	2·80		
11	18	0	3·30	1·30	

Comparing the results of the above experiments with those of the simple plate-girder, and taking the deflection indicated with a load of 18 tons, which was distributed over a distance of about 6 feet on the middle of the beam, it will be found that this beam is weak; and its elasticity, although considerable, is nevertheless so imperfect as to render it inadmissible for the support of great loads, whether proceeding from a dead weight, or one in motion rolling over its surface.

Had the centre plate been so constructed as to have angle-irons riveted in the form of flanches, similar to the plate-beam described at pp. 108 and 110, the timbers on each side would then have been useful in preventing lateral flexure; but they would not have contributed in any great degree to the vertical bearing-powers of the beam.

These defects are the more apparent in the compound construction of iron and wood, from the position of the iron plate and the difference in quality, as well as the resisting powers of the material. Where they are united in this form they can never exert, at one and the same time, those duly proportionate powers of resistance which in homogeneous material is sure to be fully developed, and calculated to exhibit its full powers of resistance.

No V

The Bridge over the Rhine at Cologne.

Letter to Baron Humboldt.

'My dear Baron Humboldt.—I gather from an article which has recently appeared in the *Times* newspaper, and from a communication which his Excellency M. Van der Heydt has honoured me with, that a most unfortunate decision has been come to by the authorities at Berlin, with reference to the important structure by which it is intended to connect the opposite banks of the Rhine at Cologne. It having been my good fortune to have been consulted, many months ago, on the subject of this important bridge, and to have visited Berlin for the purpose of submitting my proposals, I hold it due to the warm recommendation which emanated from our excellent friend in London, the Chevalier Bunsen—to the lively interest you manifested in favour of the object of my journey, and also to the gracious approval expressed by his Majesty the King of Prussia in person—to make known as widely as possible the insuperable objections which, in my opinion, attach to the limited programme which has recently been issued from the bureau of the Minister of Public Works.

'So far as words can be allowed to convey an intimation of a genuine conviction, M. Van der Heydt acknowledged at the palace on the 1st of November last, that no structure should ever be allowed to cross the Rhine which was not calculated to meet with perfect security the utmost requirements of the most extended traffic, and the possible contingencies of great military operations. Your own enlarged conceptions at once prompted you to acknowledge that the design (which at that time had received the sanction of the authorities) was totally unfit for these purposes, and to admit that a suspension bridge, owing its strength to a flexible catenary, was inadequate to the transport of heavy weights. But when I submitted the results which had been accomplished in this country by the judicious application of a material until recently untried in such structures—when I announced the successful realisation of one of the boldest conceptions of modern times—when I stated that tidal streams, such as the river Conway and the Menai Straits, had been crossed by solid and unyielding bridges of enormous span, which were capable nevertheless of sustaining ten times the greatest possible strain that the heaviest railway traffic could in practice subject them to—when I had shown that this new principle of construction was peculiarly adapted to surmount the numerous difficulties which the passage of the Rhine offers, by requiring very few and comparatively small piers in the stream, and thus allowing of the passage of large timber rafts in the summer, and offering the least possible resistance in times of floods and at the breaking up of the ice in the winter; and, above all, when a structure so much superior could be erected and fixed at an outlay considerably below that which had been demanded for a very imperfect one, I confess I was not prepared to find the minister of an enlightened and powerful people asking for the assistance of the world at large to perpetuate a scheme, unworthy of Prussia, unworthy of the practical scientific knowledge of the age, and in opposition to the deliberate and carefully weighed opinion of Science's greatest ornament.

'Pardon me for the warmth with which I address to you this remonstrance, but I feel that your unwearied exertions and the friendship which you unreservedly testified to me, call upon me to urge, as forcibly as I can, the retracing of the unfortunate steps already taken. We live in times of progress; a scientific discovery or a practical improvement of any kind cannot be confined to a particular locality or to one country; it becomes at once the property of all. This community of knowledge, the most powerful destroyer of national prejudices and antipathies, as it is the surest foundation for general and permanent peace and good-will, must ride over and bear down individual ignorance and petty bureaucratic objections. Punctuality and rapidity in our intercommunications have become almost essentials of our existence, and in this manner all Europe may be said to be interested in the completion of that railway system which will traverse the Prussian dominions from one extremity to the other.

'And now let me point out the lamentable imperfections which characterise the minister's programme, and the limitations and requirements which will effectually trammel the efforts of men of genius, and deter those of experience and reputation from entering at all upon the competition.

'It is an express condition of the scheme that the railway communication is not to be continuous, and the public will therefore continue to suffer the annoyance and inconvenience of considerable delays; for it may be safely said that the proposal of disintegrating a train at one terminus and drawing it across to the other by men or horses, bit by bit, and hour by hour, will offer equal if not greater obstacles to a rapid journey than the existing system does. How much better would it be that the bridge should embody within itself such elements of strength and durability as would afford at all times, and in all seasons, a safe transit by those means of locomotion which constitute the wonder and glory of the age? Instead of such a permanent and substantial structure, will the Prussian government sanction the erection of one, the feeble and rickety constitution of which would shudder at the very sight of a locomotive? Surely not! Public opinion must step in and forbid it. What is wanted is a bridge to connect the existing railways, not one that will permanently separate them.

'But again: it is stated that the difference between the levels of the existing railways and that required for the roadway of the intended bridge is too great to be overcome by the locomotive within a short distance of length. This objection is purely imaginary, for I can state, from personal examination, that the necessary gradient would not be so heavy as several which are worked with great ease in this country. Besides, on the left bank of the Rhine, the terminus of the Aix-la-Chapelle line is at the right level, and that on the side of Deutz may without difficulty be reached by an easy gradient of less than 1 in 100.

'Without meaning the slightest disrespect to the author of the design for the chain bridge, I must repeat my firm and deliberate conviction that it would prove an incomplete, an unsatisfactory structure. A permanent, inflexible, durable, and handsome bridge of enormous strength, (the breaking-weight of the bridge I proposed, with the span of 310 feet, was equal to 6,000 tons, or 120,000 cwts., equally distributed over each span, giving as the ultimate strength of the bridge with four spans, 24,000 tons, or 480,000 cwts.) adapted, by arrangements which I have now in progress of execution for similar purposes in this country, to give every possible facility to the navigation of the river, calculated to carry across the heaviest railway train at any speed, and which you might cover with the most powerful ordnance from end to end, may be erected at Cologne within the sum which has been demanded for the chain bridge. These statements are not the imaginings of a sanguine mind; but their accuracy may be corroborated by numerous examples of a similar character which have been erected in this country.

'If, then, the determination of the Minister of Public Works to erect a chain bridge cannot be shaken, I confidently anticipate that such an event will not be allowed to pass by without a strong protest on the part of those who are in advance of the knowledge and judgment displayed by the authors of the invitation which has been issued to the engineering world.

'My letter has attained a much greater length than I at first anticipated. My anxiety to forward your own forcibly expressed views on the subject of a fixed bridge must be my apology for it.

'With an expression of my profound esteem, and with best wishes for your continued good health,

'Believe me to remain, my dear Baron Humboldt, your very faithful and very obedient servant,

'WILLIAM FAIRBAIRN.

'MANCHESTER, *April* 15th, 1850.'

INDEX.

AIRY, Mr. G. B., Astronomer Royal, his paper on the law of strains in the interior of beams, quoted, 268

Anthracite iron, rigidity and strength of, 69

Arches of factories, mode of building, 155

— use of thin iron plates instead of, 159

Arches, cast-iron, powers of resistance of, 203

— mode of suspending a roadway beneath a large cast-iron arch, 203

BEAMS, cast-iron, for supporting the floors of buildings, 1

— fire-proof cotton mill of Messrs. Philips and Lee at Manchester, 2

— sectional dimensions at the middle, 3

— longitudinal section of portions of the basement and first stories of the mill, 4

— Mr. Hodgkinson's experiments as to the strength of iron beams, 5, 9–24

— Mr. Fairbairn's experiments, 5, 6

— Mr. Tredgold's, 8

— rule for strength of cast-iron beams, 23

— Mr. Tate's formula, 24 *note.*

— table of the weight, area, section, distance between the supports, breaking weights, &c., of beams in use before Mr. Hodgkinson's experiments, 25

— the best form of section, 25

— girder beams of 76 feet span in one casting, 26

Beams of cast iron—*continued.*

— accidents which have occurred to cast-iron beams, 26, 27

— compound or trussed beams or girders of cast-iron, 27

— remarks on the value of the tension-rods, 27

— tables of experiments giving exact comparative view of the physical and mechanical difference between cast and wrought iron, 29

— law relative to the forces requisite for producing equal elongations in cast-iron and wrought-iron bars, 30

— total ultimate elongation of wrought-iron as compared with that of cast-iron, 31

— difference of the permanent sets of cast-iron and wrought-iron, 32

— mean elongation of cast and wrought-iron bars at 90° temperature, 32

— determination of the best adjustment of the tension of the truss-rods, 32

— — First, when the truss-rods have no strain upon them at the time when the beam is unloaded, 32

— — Second, when the truss-rods are screwed up so as to give them a tension of 8 tons per square inch, and the half length of the beam is 10 feet, 34

— — Third, when the tension given to the truss-rods is 2½ tons per square inch for the cast-iron, and 8 tons per square inch for the wrought-iron, 34

— results of these experiments, 35

— approximate rule for calculating the strength of a truss-beam, 36

BEA

Beams of cast iron—*continued.*
- — comparison of cost of different forms of beams, 37
- — trussed beams, where the rods are attached to points above the line of the top flange, 38
- — experiments to determine what advantages, if any, are derived from wrought-iron trusses applied to cast-iron beams as an additional support, 40
- — summary of results, 43
- — general remarks relative to cast-iron beams, 44
- — on the influence of time on the strength of cast-iron bars, 47
- — on the influence of temperature in producing the same result, 54
- — on the comparative strength of iron after successive re-meltings, 56
- — mixtures of cast-iron, 62
- — girders made of toughened iron, 64
- — results of experiments made upon unprepared cast-iron and toughened cast-iron, 66
- — results of a mixture of cast and wrought iron, 66, 67
- — combinations or mixtures of iron, 68
- — experiments on anthracite iron, 69
- — experiments on the transverse strength of iron, made under the direction of Robert Stephenson, Esq. 70
- — tensile and compressive strength of various descriptions of iron, 72
- — experimental inquiry on the strength, &c., of cast-iron bars, 73, 74
- — rule to find the breaking weight of rectangular bars generally, 75
- — results of experiments made by order of the United States' Government, 75
- — difference in weight of cast and wrought iron beams, 82
- — remarks on the mode of testing cast-iron beams, 172
- — resisting powers of beams, 175

Beams of wrought iron, for supporting the floors of buildings, and for other purposes, 78

BEA

Beams of wrought iron—*continued.*
- — the box-beam, 78
- — the plate-beam, 78
- — comparative resisting powers of the two beams, 78
- — particulars of beams requiring a centre column with a distance of 30 feet between the bearings, 81
- — beams with cellular tops, 83, 84
- — beams with the bottom flange rolled along with the plate, and the top of T iron riveted to the vertical plate, 85
- — sectional dimensions of beams exhibited at the Paris Exposition in 1855, 85
- — systems of fire-proof flooring in use on the Continent, 86 *note*
- — construction of beams of large span and considerable strength, 87
- — beams formed by joining their lengths together, 88
- — experiments on the strength, &c., of wrought-iron beams, 89
- — experiments on the transverse strength of malleable iron rectangular tubes, 90—110
- — summary of results of experiments on rectangular tubes, 111
- — comparative strengths of rectangular tubes, indicated by the value of C, 112
- — comparative weights and strengths of rectangular tubes, 113
- — comparative strengths of rectangular beams, indicated by the value of C, 113
- — experiments to determine the strength and security of open trellis girders, 114 *et seq.*
- — determination of the constant for trellis beams, 118, 121
- — instance of the weakness and insufficiency of the trellis girders, 120
- — half-elevation and cross section of the girder which is employed in a bridge on the Ulverstone and Lancaster Railway, 121, 122
- — results of experiments with this girder, 123
- — experiments with a lattice girder, 123—128

BEA

Beams of wrought iron—*continued.*
— comparative constants of the tubular or box girder, the plate girder, and the lattice girder, 124
— Mr. Unwin's calculations of the horizontal strains to which the different parts of the top and bottom of the girders of the Tay Viaduct were subjected from the centre of the span in the direction of the piers at each end, 128
— strains on the double web of lattice work on each girder, 132
— lattice work of small span, 135
— strains on lattice bars under tension, 136
— Mr. Hodgkinson's parabolic form of beam for warehouses and factories, 145
— resistance of various kinds of beams to impact, 146, 147 *note*
— details of the application of wrought-iron beams for the support of the floors of buildings, 152
— details and arrangement of the wrought-iron beams used in the construction of the sugar refinery of Messrs. Bewley, Moss, & Co., 166
— the adaptation of malleable iron beams or girders for the construction of bridges, 187
— advantages of plate girders for bridges of small span, 205
— facts proving the superior efficiency of wrought-iron beams, 287
— the box and the plate beam, 298
Abstract of the paper of the Astronomer Royal on the law of strains on the interior of beams, quoted, 268
Blackburn tubular girder bridge, results of experiments on, 234
Blood, Mr., his mathematical analysis of the strength of a Warren girder bridge, 210
Bowstring Bridge, description of the, 205, 266
Brancker, Mr., his fire-proof warehouses in Liverpool, 143
Brick, use of, in bridge-building, 198
— Mr. Latimer Clarke's experiments on the resisting powers of brickwork, 199
Bridges, on the adaptation of malleable-iron beams or girders for the construction of, 187
— the three classes of bridges now in use, 187
— the earliest forms of bridges, 188
— materials of which bridges are composed:—
—— timber, 190
—— stone, 193
—— brick, 198
—— wrought and cast iron, 199
— details of Mr. Pritchard's cast-iron bridge over the Severn at Coalbrookdale, 201
— and of Mr. Burdon's bridge over the Wear at Sunderland, 201
— Messrs. Telford and Douglas's designs for a colossal cast-iron bridge to replace old London Bridge, 202
— Mr. Rennie's plan for a cast-iron bridge over the Menai Straits, 202
— Mr. Telford's chain suspension bridge over the Menai Straits, 202
— the cast-iron bridge over the Thames at Southwark, 202
— that over the Neva at St. Petersburgh, 202
— powers of resistance of the cast-iron arch, 203
— employment of horizontal girders for bridges of short span, 203
— mode of suspending a roadway beneath a large cast-iron arch, 203
— the Britannia Tubular Bridge, 204
— principle of support in tubular bridges, 205
— advantages of plate girders for bridges of small span, 205
— the Bowstring Bridge, 205
— the lattice form of girder bridge, its advantages and disadvantages, 206
— details of the Newark Dyke lattice Warren girder bridge on the Great Northern Railway, 207
— mathematical analysis of the strength of a Warren girder bridge, 210
— experiments for determining the form of tube which gives the greatest power of resistance with the least quantity of material, 215—220

Bridges—*continued.*
— formation of the bottom of a tubular girder, 223
— experiments on chain riveting, 224
— formation of the sides of a tubular girder, 226
— principle on which to estimate the strength of wrought-iron tubes, 229
— the Torksey Tubular Bridge, 229
— tables, exhibiting the strength, proportions, and other properties of girders recommended in tubular bridges, 232
— force of impact and effect of vibration on tubular bridges, 233
— mode of testing tubular girder bridges, 235
— Mr. Tate's investigation of formulæ relative to rectangular, cylindrical, and elliptical tubes, 236–243
— investigation of general formulæ applicable to the Torksey Bridge, 244–252
— on the construction of tubular bridges of single span, 254
— details of that of Mr. J. Mitchell over the Spey, 254
— plate bridges, for small spans, 262
— various forms of iron bridges, 266
— attempts to connect a chain suspension bridge with an unyielding rigid structure, 267
— account of the proposed tubular and tubular girder bridge for crossing the Rhine at Cologne, 275
Britannia Tubular Bridge, 204
Brunel, Mr., his tubular bridge at Chepstow, 266
— his adoption of the cellular system in the construction of the Great Eastern, 266
Burdon, Mr., his iron bridge over the Wear at Sunderland, 201
— details of this bridge, 201, 202

CHAIN-RIVETING, advantages of, 224
— experiments with single plates, with double covers over the joint, 224
— experiments with double plates and a single cover over the joint, 225
Clarke, Mr. Latimer, his experiments on the resisting powers of sandstone and limestone, 194, 195
— and on the resisting powers of brickwork, 199.
Coalbrookdale, the first cast-iron bridge in England erected at, 2 *note*, 201
Columns, strength and proportion of, in fire-proof buildings, 149
— strength of columns of cast-iron, 150
— form of column and base-plate for fire-proof buildings, 158–163
Compound or trussed cast-iron beams or girders. *See* Beams
Cotton mills, extent and amount of work done in that of Messrs. Whittaker and Brothers, of Ashton-under-Lyne, 170
— those of Messrs. Fielden and Brothers and of Messrs. Ashton and Sons, 171
— details of the Saltaire mills, 177—186
— report on the causes of the fall of the cotton mill at Oldham, 287

DARGAN, Mr. William, his Great Exhibition building in Dublin, in 1853, trellis girders of, 114

EXHIBITION, Great, of Dublin, trellis girders of, 114

FAIRBAIRN, Mr., his experiments as to the strength of cast-iron beams, 5
— results of his experiments, 6, 7
Fire-proof warehouses, on the construction of, 137
— report as to the expediency of erecting all the new warehouses in Liverpool with fire-proof materials, 137
— enumeration of principles upon which warehouses should be constructed in order to give perfect security, 139
— choice of non-combustible materials, 140
— every opening or crevice communi-

Fire-proof warehouses—*continued.*
eating with the external air should be closed, 141
— isolated stone staircase, 141
— Mr. Jones's apparatus for extinguishing fire, 142
— different warehouses, in a range of stories, should be divided by strong partition walls, 143
— sufficient strength of the iron columns, beams, and brick arches, 144
— Mr. Hodgkinson's, the strongest form of beam, as applied to fire-proof buildings, 145
— strength and proportion of the columns, 148–151, 158–169
— importance of the tie-rods, 148
— solitary instance of the melting of iron columns by intense heat, 151 *note*
— details of the application of wrought-iron beams for the support of the floors of buildings, 152
— brick arches of warehouses and factories, 155
— use of thin iron plates instead of arches, 159
— the author's fire-proof building erected for Messrs. Bewley, Moss & Co., 166
— floors formed with light wrought-iron joists resting on the main beams, 170
— details of the fire-proof cotton mill of Messrs. John Whittaker & Co., of Ashton-under-Lyne, 170 *et seq.*
- - details of the Saltaire mills, 177 *et seq.*
— paper on some defects in the principle and construction of fire-proof buildings, 293
Floors of buildings, cast-iron beams for supporting the, 1
Fox and Henderson, Messrs., their iron bridges, 266

GIRDERS, cast-iron. *See* Beams
Gray, Mr., accident at his mill at Manchester, 27
Greeks, buildings of the, 138

HODGKINSON, Mr., his experiments as to the strength of iron beams, 5, 9–24
— his treatise on the strength of pillars of cast iron and other materials, 149
— table of his experiments on the compressive strength of wood, 191
Humboldt, Baron, author's letter to, respecting the proposed tubular and tubular girder bridge over the Rhine at Cologne, 300

IMPACT, force of, on tubular bridges, 233
Iron, physical and mechanical difference between cast and wrought, 29
— tables of experiments giving an exact comparative view of these properties in cast and wrought iron, 29
— law relating to the forces requisite for producing equal elongations in cast-iron and wrought-iron bars, 30
— comparative breaking weights of wrought and cast iron, 31
— uncertainty respecting the 'shrinkage' or contraction of metals during the process of cooling, 45
— rupture from various causes, 45
— defects which lie concealed under the surface of a casting, 46
— effects of time and temperature on the strength of cast iron, 47–56
— on the comparative strength of iron after successive re-meltings, 56
— Mr. Morries Stirling's processes for toughening cast iron, 64
— cupola and air furnace, 65
— Mr. Lillie's experiments with a mixture of cast and wrought iron, 67
— tables of experiments on various properties of cast iron, 70–74
— cast iron compared with malleable iron in bridge-building, 200
— the first cast-iron bridge, that of Mr. T. F. Pritchard over the Severn, at Coalbrookdale, 2, 201

JON

JONES, Mr., of Wallshaw, his apparatus for extinguishing fire, 142

LATTICE girders of wrought iron. *See* Beams of wrought iron
— advantages and disadvantages of lattice girder bridges, 204 *et seq.*
Liverpool, report as to the expediency of erecting all the new warehouses in, of fire-proof materials, 137
— Mr. Brancker's fire-proof warehouse in Dublin Street, 143
London Bridge, 198

MANCHESTER, Art Treasures Exhibition building at, 120
Menai Straits, Mr. Telford's chain suspension bridge over the, 202
Mitchell, Mr. J., details of his bridge over the Spey, 254

NEVA, details of the colossal iron bridge over the, at St. Petersburgh, 202
Newark Dyke lattice bridge, details of, 205
Niagara, wire railway bridge at, 267

OLDHAM, report on the causes of the fall of the cotton-mill at, 27, 287
Owen, Mr., his experiments on Mr. Morries Stirling's toughened iron, 64

PARTITION walls dividing houses, their importance, 143
Philips and Lee, Messrs., their fire-proof cotton-mill at Manchester, 2
Plate bridges, for small spans, 262
Pole, Mr., his investigation of general formulæ applicable to the Torksey Bridge, 244
Pritchard, Thomas Farnolls, the inventor of iron bridges, 2
— his cast-iron bridge over the Severn at Coalbrookdale, 2, 201
— details of this, the first cast-iron bridge, 201

TAB

RAILWAY bridges, varieties of, 189
— best form of, 189, 190
Rennie, Mr., his designs for a cast-iron bridge over the Menai Straits, 202
Rhine, account of the proposed tubular and tubular girder bridge for crossing the, at Cologne, 275
— letter to Baron Humboldt respecting this bridge, 300
Romans, buildings of the, 138

SALFORD, accident in Mr. Nathan Gough's mill at, 27
Saltaire mills, position of the, 178
— extent of the buildings, 178
— details of construction, 179
— gas and water supply, 184
— capabilities of the Saltaire mills, 186
Severn, Mr. Pritchard's cast-iron bridge over the, at Coalbrookdale, 201
Southwark iron bridge, details of, 202
Spey, Mr. Mitchell's single-span tubular bridge over the, 254
Stirling, Mr. Morries, his processes of toughening iron, 64
Stone, adaptability of, for bridge-building, 193
— Mr. Latimer Clarke's experiments on the resisting powers of sandstone and limestone, 194, 195
— experiments to determine the force required to crush different kinds of stone, 196
— experiments to ascertain the amount of water absorbed by different kinds of stone, 197

TABLES:—

Tables relating to Cast and Wrought Iron Beams.
— Of the weight, area, section, distance between the supports, breaking weights, &c., of beams in use before Mr. Hodgkinson's experiments, 25
Table I. Mean elongations by tensile forces within the limits requisite to

Tables—*continued.*
rupture cast-iron, viz., about 7½ tons per square inch of the transverse section, 29
Table II. Mean elongations and sets, with tensile forces equal to two thirds of the forces requisite to produce rupture in each case respectively, 30
— III. Mean values of the tensile forces requisite for producing equal elongation in cast-iron and wrought-iron bars 10 feet long, with the corresponding sets, 30
— IV. Ultimate elongations, the cast-iron being loaded with 7½ tons per square inch, and wrought-iron with 24 tons per square inch, 31
— V. Permanent set of bars, expressed in parts of their elongation, 32
— VI. Mean elongation of cast-iron and wrought-iron bars 10 feet long, by an increase of 90° temperature, 32
— Experiments on a beam of the strongest section, 4 feet 6 inches between the supports and two truss-rods, each one-half inch diameter to the bottom and ends, with the broad flange below, 40
— The same beam, with the broad flange below, and without truss-rods, 4 feet 6 inches between the supports, 41
— The same beam, with the broad flange above and two truss-rods, three quarters of an inch diameter, 4 feet 6 inches between the supports, 41
— The same beam reversed, with the broad flange uppermost and without truss-rods, 4 feet 6 inches between the supports, 42
— summary of results, 43
— Coed-Talon rectangular bars (4 feet 6 inches between the supports), loaded with different weights for determining the changes, if any, which take place during indefinite periods of time; the mean of the breaking weights of each description of iron having been previously ascertained to be, for the cold blast 508 lbs., and for the hot blast 484 lbs., 50
— results on bars 2 and 7, loaded with 336 lbs., 50
— results on bars 3 and 8, loaded with 392 lbs., 51
— results on bars 4, 5, 9, and 10, loaded respectively with 448 lbs., 51
— summary of mean results of the influence of time on the strength of cast-iron bars, 53
— comparative strength and power to resist impact of the Coed-Talon hot and cold blast irons at various temperatures, 55
— transverse strengths, 55
— power to resist impact, 55
— modulus of elasticity in pounds for a base of 1 inch square, 56
— results of experiments on the transverse and crushing strength of re-melted iron, 58
— Major Wade's experiments on the same point, 59
— method of mixing iron from different furnaces, and results of different mode of treatment, 63
— mixtures producing the strong specimens obtained by this method, 63
— experiments made as to the comparative strength of unprepared cast iron and toughened cast iron, 66
— experiments as to the advantages of a mixture of cast and wrought iron, 66
— Mr. Morries Stirling's iron, 66
— Blaenavon iron, No. 2, 67
— Mr. Lillie's experiments, 67
— experiments on the transverse strength of iron, 70
— tensile and compressive strength of various descriptions of iron, 72
— experimental inquiry on the strength, &c., of cast-iron bars, 73, 74
— results of experiments made by order of the United States' Government, 76
— transverse strength of malleable-iron rectangular tubes, 90–110
— summary of results of experiments on rectangular tubes, 111

Tables—*continued.*
- — comparative strengths of rectangular tubes indicated by the value of C, 112
- — comparative weights and strengths of rectangular tubes, 113
- — comparative strengths of rectangular beams, indicated by the value of C, 113
- — experiments to determine the strength and security of open trellis girders, as shown in the diagram, fig. 49, 116
- — experiment to determine the strength of open trellis girders similar to that shown in fig. 51, 120
- — experiments with a lattice girder, submitted to a transverse strain, 127
- — strains on the double web of lattice work on each girder of the Tay Viaduct, 133
- — strains on lattice bars under compression, 134
- — strains on lattice bars under tension, 136

Tables on the Strength of Wood.
- — compressive strength of wood, 191
- — tensile strength of wood, according to Muschenbroeck and Barlow, 192

Tables on the Properties of Stone.
- — resisting powers of sandstone, 194
- — resisting powers of limestone, 195
- — force required to crush different kinds of stone, 196
- — amount of water absorbed by different kinds of stone, 197.

Table on the Properties of Brickwork.
- — Mr. Latimer Clarke's experiment on the resisting powers of brick-work, 199

Tables for determining the Form of Cells in Tubular Work.

Table I. Experiments made to determine the resistance of plates (or bars) of wrought-iron to a force of compression; the plates being placed in a vertical position, with their ends made perfectly flat, so as to be well bedded against two parallel and horizontal crushing surfaces, 217
- — II. Resistance of rectangular tubes, all 10 feet long, to a force of compression in the direction of their length, 218
- — III. Resistance of circular tubes, all 10 feet long, to a force of compression in the direction of their length, 219
- — Table of experiments upon a cell of the same dimensions as those in the Conway and Britannia bridges, 220

Tables exhibiting the Strength, Proportions, and other Properties of Girders recommended in Tubular Bridges.
- — table showing the proportions of tubular girder bridges, from 30 to 150 feet span; where the depth of the girder is $\frac{1}{13}$th the span, 232
- — table showing the proportions of tubular girder bridges, from 160 to 300 feet span; where the depth of the girder is $\frac{1}{15}$th the span, 232
- — calculated deflection of the Torksey Bridge under the specified load, 251

Tate, Mr., his formula for determining the strength of cast-iron beams, 24 *note*
- — on the best form of cell for the top of a tubular bridge, 219
- — his investigations of formulæ relative to rectangular tubes, 236
- — formulæ relative to cylindrical tubes, 240
- — and to elliptical tubes, 241

Tay railway bridge at Dunkeld, experiments with the model lattice girders of the, 123–128
- — Mr. Unwin's calculations of the horizontal strains to which different parts of the top and bottom of a lattice girder of the Tay Bridge were subjected, 128–136

Telford, Mr., his designs for a colossal cast-iron bridge to replace old London Bridge, 202

Telford, Mr.—*continued.*
— his chain suspension bridge over the Menai Straits, 202
Temperature, effects of, upon the strength of cast-iron, 54
Tie-rods, value of, in fire-proof buildings, 148
Timber, use of, in building bridges, 190
— Mr. Hodgkinson's experiments, 191
— table of the compressive strength of wood, 191
Time, effects of, upon the strength of cast iron, 47
Torksey tubular bridge, details of the, 229, 230
— investigation of general formulæ applicable to the, 244
Tredgold, Mr., on the value of cast iron, quoted, 1
— his experiments on the strength of cast-iron beams, quoted, 8
Trellis girders, open. *See* Beams of wrought iron
Truss-beams. *See* Beams or Girders
Tubes, experiments on the transverse strength of malleable-iron rectangular, 90. *See* Beams of wrought iron
Tubular and tubular girder bridges, 213
— experiments for obtaining the greatest power of resistance to compression with the least quantity of material, 213–220.
— the cellular system in nature, 221
— formation of the bottom of a tubular girder, 223
— formation of the sides of a tubular girder, 226
— principle on which to estimate the strength of wrought-iron tubes, 229
— strength, proportion, &c., of girders for tubular bridges, 232
— force of impact and effect of vibration on tubular bridges, 233
— mode of testing tubular girder bridges, 235
— Mr. Tate's investigation of formulæ relative to rectangular tubes, 236
— formulæ relative to cylindrical tubes, 240
Tubular and tubular girder bridges—*continued.*
— and to elliptical tubes, 241
— formula to express the breaking-weight of a tube, as compared with the weight of the tube itself, 243
— continuity of girders over two or more spans, 243
— investigation of general formulæ applicable to the Torksey Bridge, 244
— calculated defection of the Torksey Bridge under the specified load, 251
— on the construction of tubular girder bridges of single span, 254
— account of the proposed tubular and tubular girder bridge for crossing the Rhine at Cologne, 275

ULVERSTONE and Lancaster Railway, half elevation and cross section of one of the girders of a bridge on the, 121, 122
United States' Government, results of experiments on the tensile strength, &c., of cast iron, 76

VIBRATION, effect of, on tubular bridges, 233

WADE, Major, his experiments on re-melted iron, 58
Warehouses, general character of, for ages, 139
— enumeration of principles upon which warehouses should be constructed, in order to give perfect security, 139
— *See* Fire-proof warehouses
Warren, Capt., his girders, 206
— their weakness, 209, 210
— Mr. Blood's mathematical analysis of the elements of strength in a Warren girder bridge, 210
Waterloo Bridge, 198
Wear, Mr. Burdon's iron bridge over the, at Sunderland, 201, 202

Wheels, bevel, one of the first made from cast iron, 2 *note*
Wilkinson, Mr. John, his cast-iron bridge at Coalbrookdale, the first in England, 2 *note*
Wilmot, Colonel, his experiments upon different kinds of mixed cast iron, 63
Wire bridge at Niagara, 267
Wye, Mr. Brunel's cellular bridge over the, at Chepstow, 266

Errata.

Page xiii line 7 *for* page 13 *read* page 23
In Appendices, *for* flanch *read* flange ; *for* storeys *read* stories
Page 290 line 3 (note) *for* ⊤ *read* ⊥ ; line 5 *for* ⊥ *read* ⊤
295 ,, 9 *for* figs. 101 & 102 *read* 117 & 118
299 ,, 10 from bottom *for* pp. 108 & 110 *read* 78 & 81.

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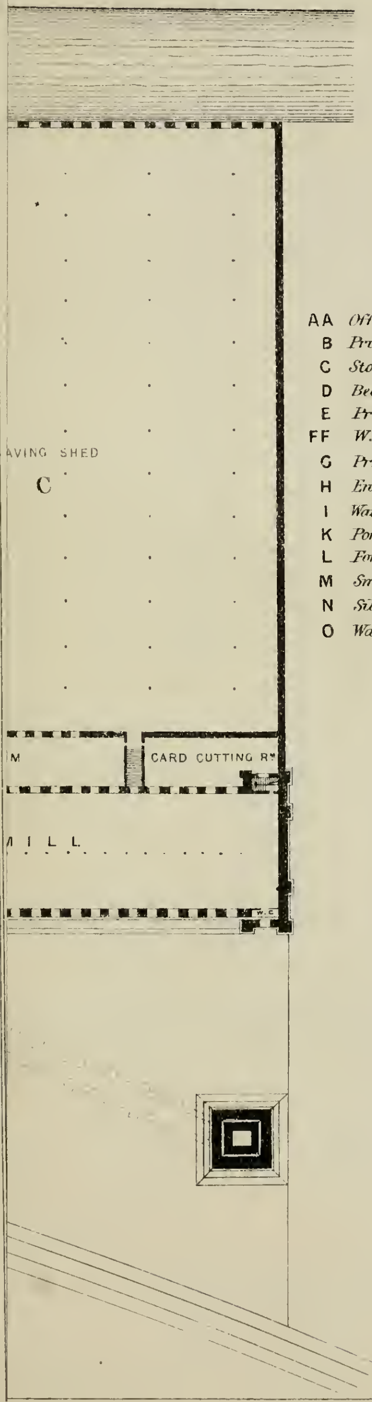

REFERENCE.

AA	*Office Buildings.*
B	*Private Dining Room.*
C	*Store Room*
D	*Bed Room.*
E	*Principal Counting House.*
FF	*W.*
G	*Private Counting House.*
H	*Entrance Lobby.*
I	*Waiting Room.*
K	*Porter's Room.*
L	*Foreman's Counting House.*
M	*Small Store Room.*
N	*Silk Warping Room.*
O	*Warp Stock Room.*

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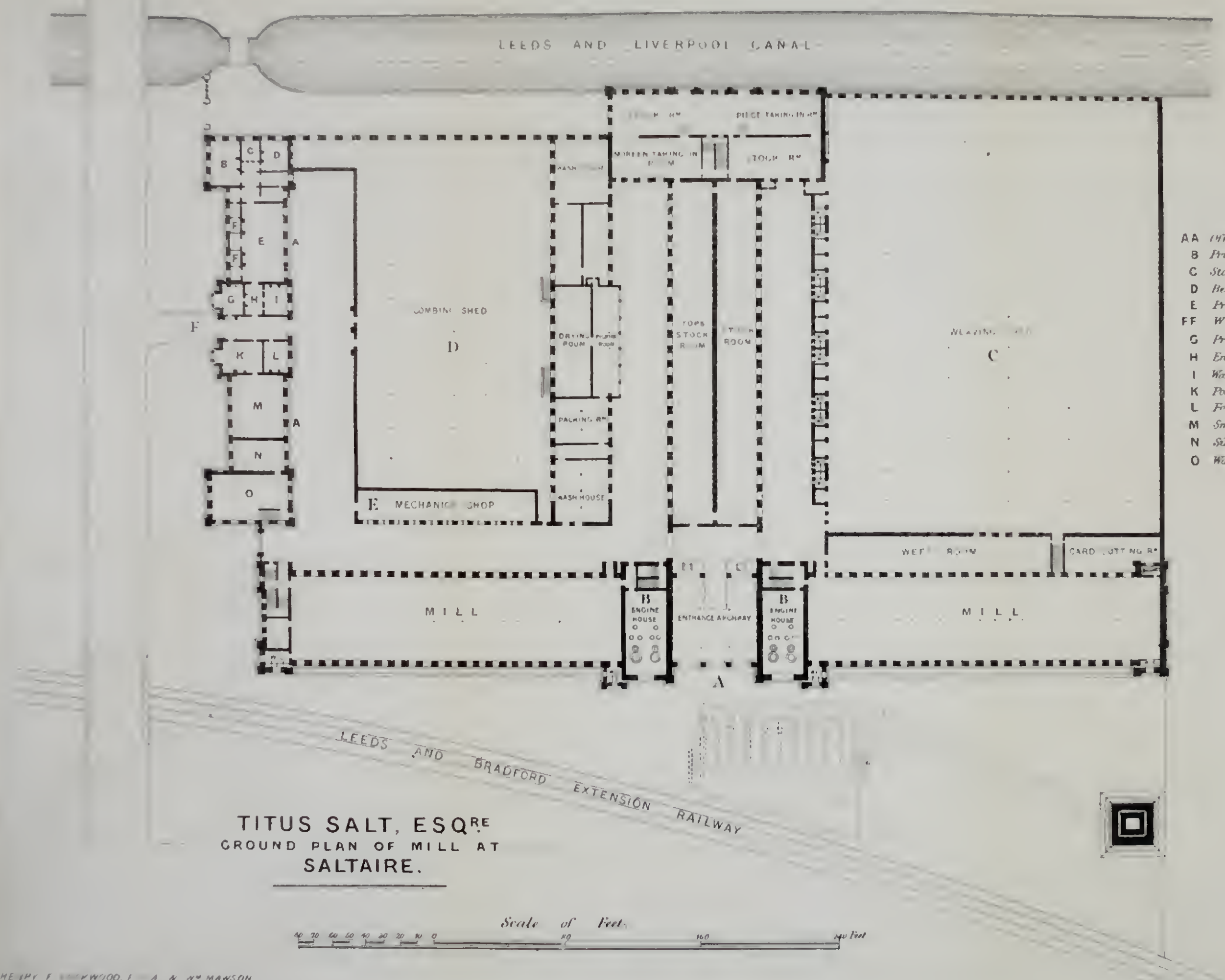

TITUS SALT, ESQre
GROUND PLAN OF MILL AT
SALTAIRE.

REFERENCE

AA *Office Buildings*
B *Private Dining Room*
C *Store Room*
D *Bed Rooms*
E *Principal Counting House.*
FF *W*
G *Private Counting House*
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I *Waiting Room*
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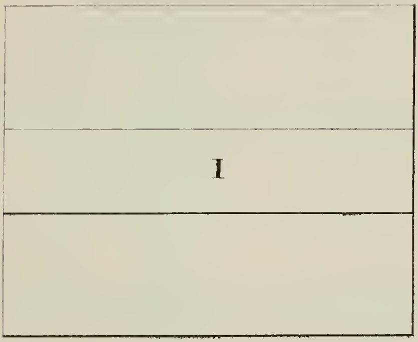

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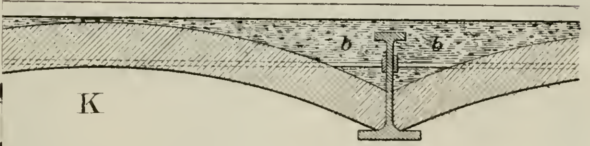

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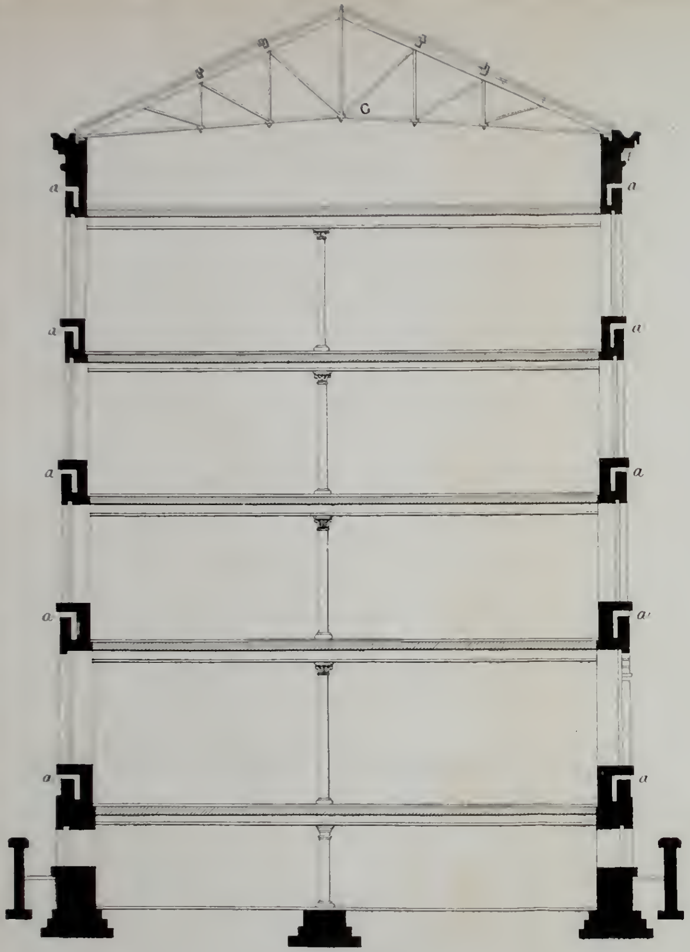

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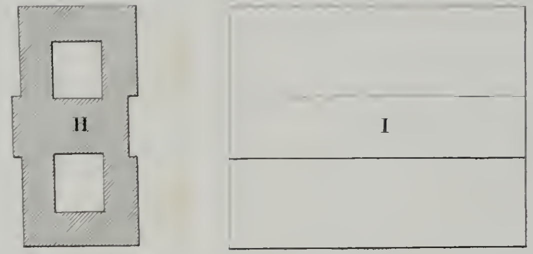

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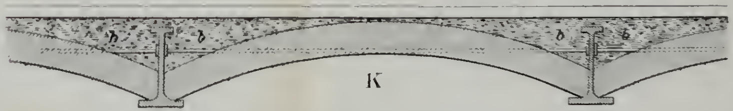

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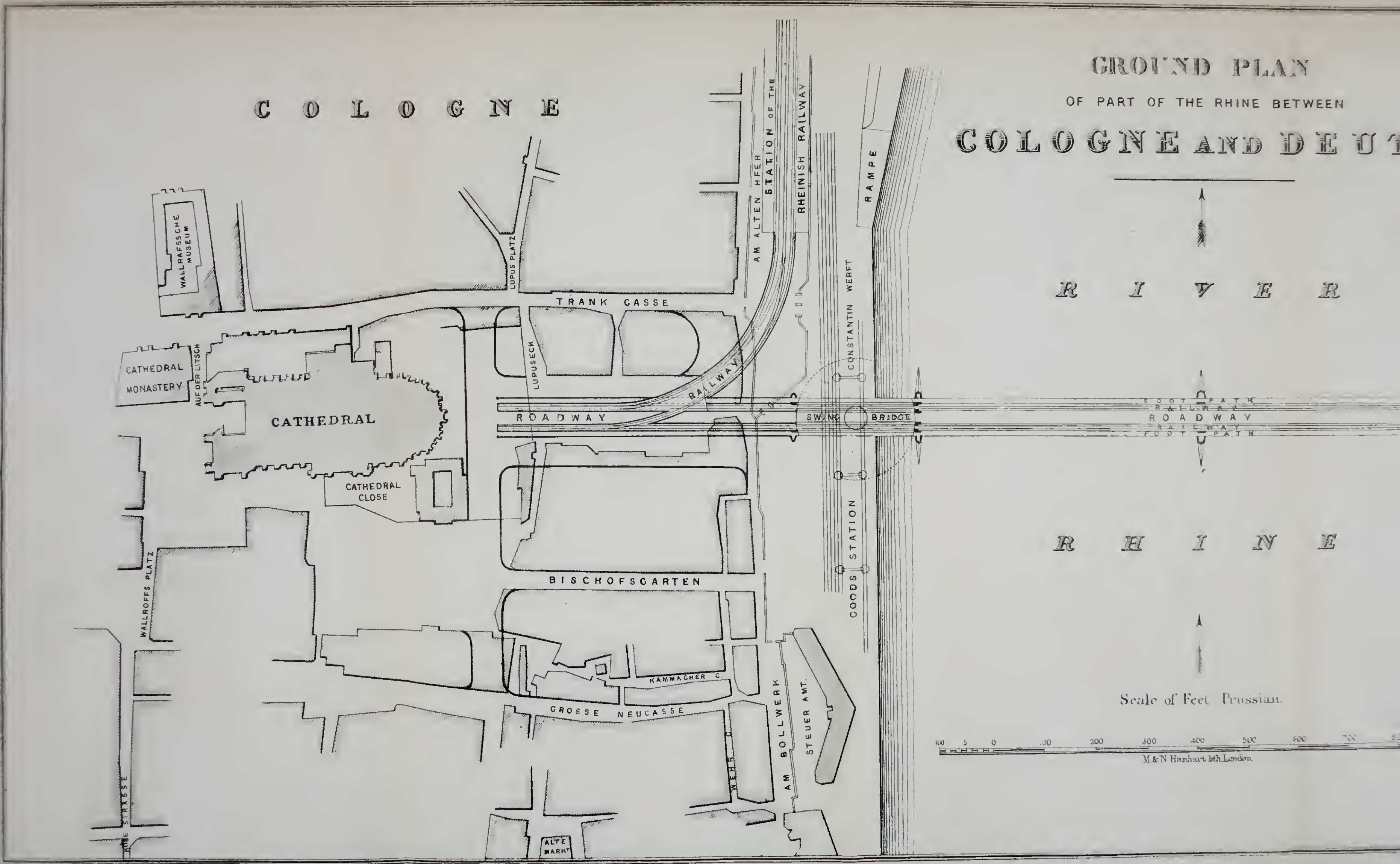

GROUND PLAN
OF PART OF THE RHINE BETWEEN
COLOGNE AND DEUT

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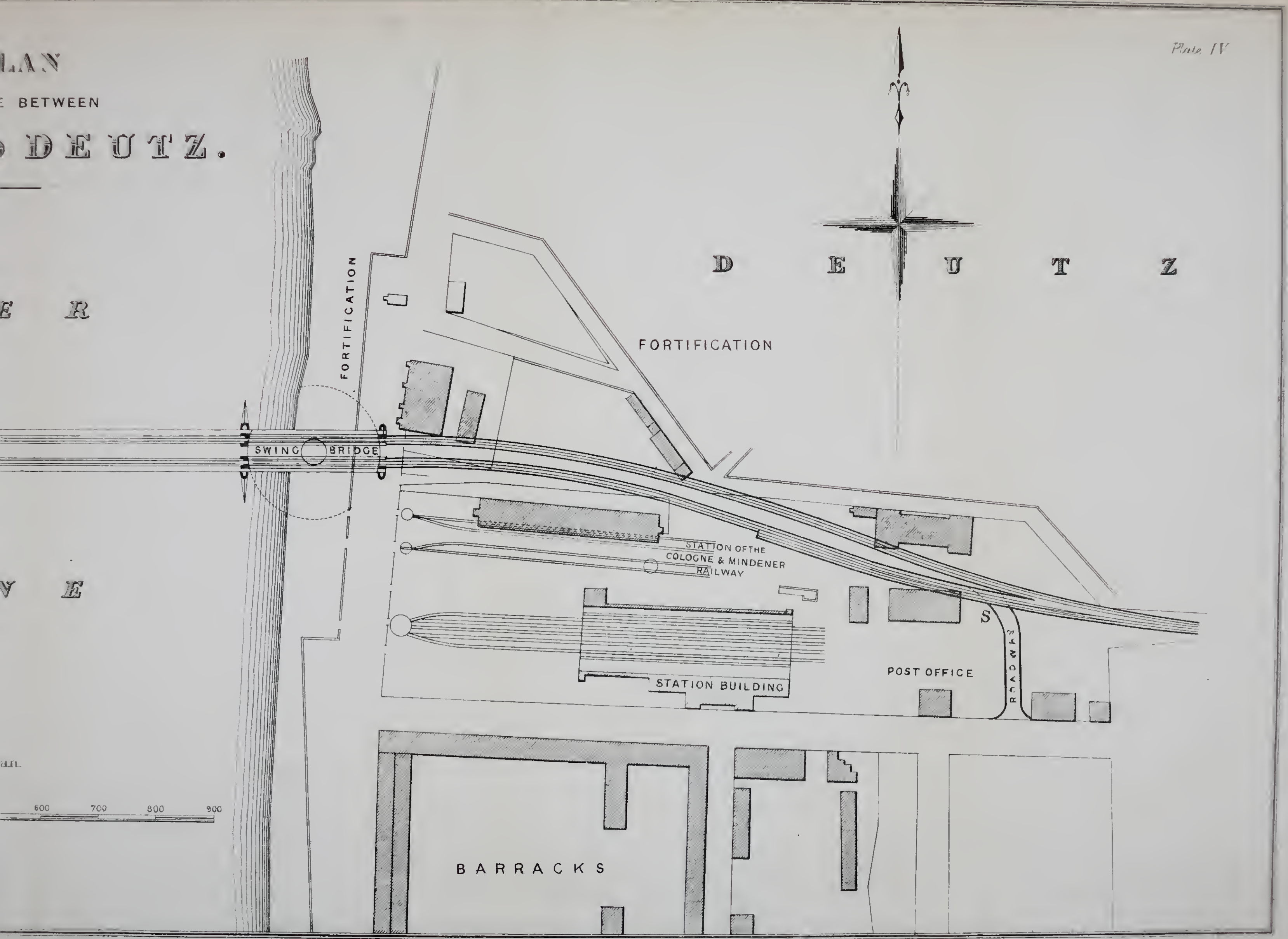
Plate IV
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FORTIFICATION
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FORTIFICATION
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POST OFFICE
STATION BUILDING
BARRACKS

PROPOSED TUBULAR GIRDER BRIDGE, FOR CR

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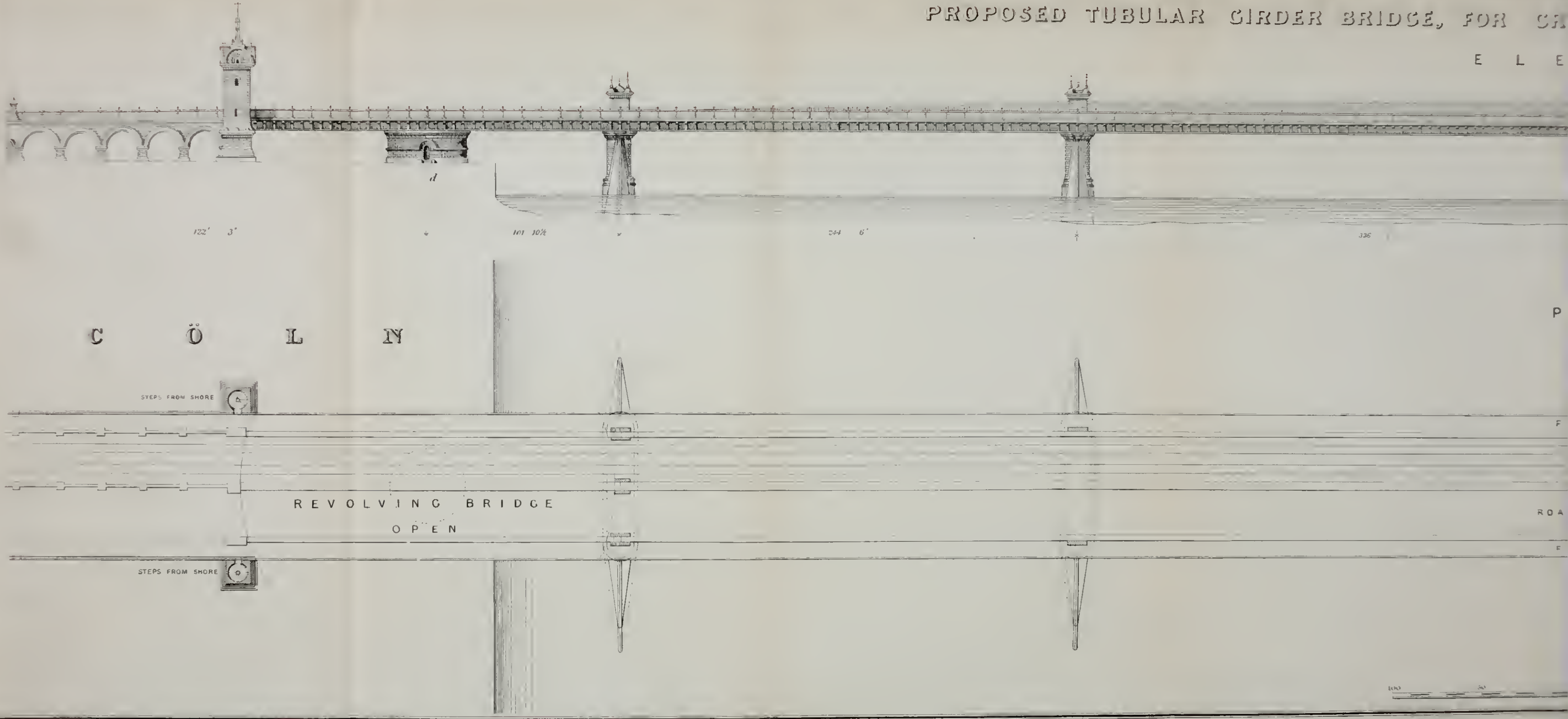

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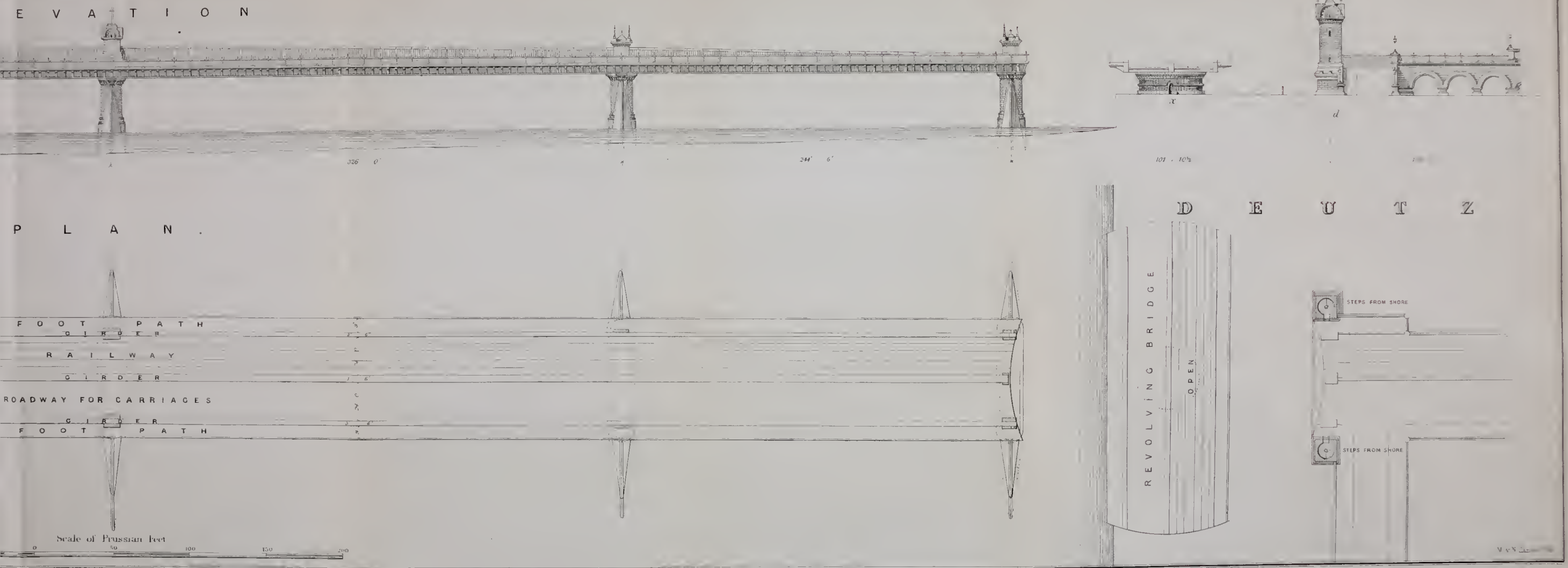

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Scale of Prussian Feet

Plate VI

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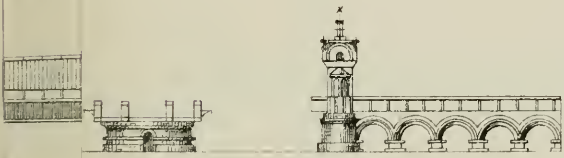

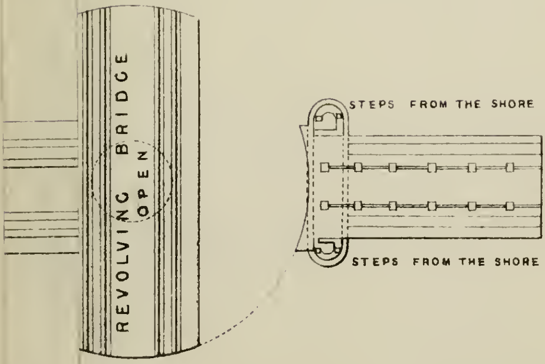

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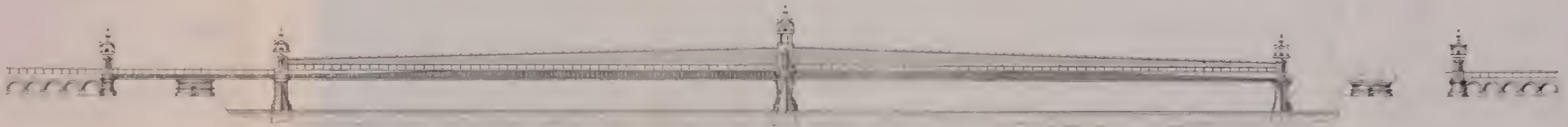
PROPOSED TUBULAR BRIDGE ACROSS THE RHINE AT COLOGNE.

ELEVATION



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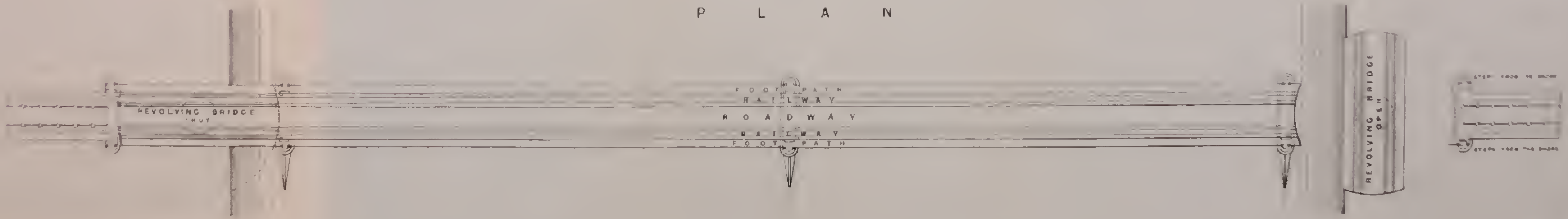

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Alcock's Residence in Japan 16
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—— on the Senses and Intellect 7
—— on the Study of Character 7
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———Guide to the Western Alps 16
———Guide to the Eastern Alps 16
Barnard's Drawing from Nature 12
Bayldon's Rents and Tillages 13
Beaten Tracks 16
Becker's *Charicles* and *Gallus* 17
Beethoven's Letters 4
Benfey's Sanskrit-English Dictionary 6
Berry's Journals 3
Black's Treatise on Brewing 20
Blackley and Friedlander's German and English Dictionary 6
Blaine's Rural Sports 19
———Veterinary Art 19
Blight's Week at the Land's End 17
Booth's Epigrams 6
Bourne on Screw Propeller 13
———'s Catechism of the Steam Engine 13
——— Handbook of Steam Engine 13
——— Treatise on the Steam Engine 13
Bowdler's Family Shakspeare 18
Bramley-Moore's Six Sisters of the Valleys 17
Brande's Dictionary of Science, Literature, and Art 10
Bray's (C.) Education of the Feelings 7
——— Philosophy of Necessity 7
——— On Force 7
Brinton on Food and Digestion 20
Bristow's Glossary of Mineralogy 8
Brodie's Constitutional History 1
——— (Sir C. B.) Works 11
Browne's Exposition 39 Articles 14
Buckle's History of Civilisation 2
Bull's Hints to Mothers 20
——— Maternal Management of Children 20
Bunsen's Ancient Egypt 3
——— God in History 3
——— Memoirs 3
Bunsen (E. De) on Apocrypha 15
———'s Keys of St. Peter 15
Burke's Vicissitudes of Families 4
Burton's Christian Church 3

Cabinet Lawyer 20
Calvert's Wife's Manual 15
Cates's Biographical Dictionary 3
Cats and Farlie's Moral Emblems 12
Chorale Book for England 11
Christian Schools and Scholars 6
Clough's Lives from Plutarch 2
Colenso (Bishop) on Pentateuch and Book of Joshua 15
Collins's Horse Trainer's Guide 19
Commonplace Philosopher in Town and Country 6
Conington's Chemical Analysis 10
——— Translation of Virgil's Æneid 18
Contanseau's Two French and English Dictionaries 6
Conybeare and Howson's Life and Epistles of St. Paul 14
Cook's Acts of the Apostles 14
Copland's Dictionary of Practical Medicine 11
Coulthart's Decimal Interest Tables 20
Cox's Manual of Mythology 17
——— Tales of the Great Persian War 2
——— Tales from Greek Mythology 17
——— Tales of the Gods and Heroes 17
——— Tales of Thebes and Argos 17
Crawley's Billiard Book 20
Cresy's Encyclopædia of Civil Engineering 13
Critical Essays of a Country Parson 6
Crowe's History of France 2
Crump on Banking, &c 19
Cussans's Grammar of Heraldry 12

Dart's Iliad of Homer 18
D'Aubigné's History of the Reformation in the time of Calvin 2
Davidson's Introduction to New Testament 14
Dayman's Dante's Divina Commedia 18
Dead Shot (The), by Marksman 19
De Burgh's Maritime International Law 20
De la Rive's Treatise on Electricity 8
De Morgan on Matter and Spirit 7
De Tocqueville's Democracy in America 2
Disraeli's Speeches on Reform 5
Dobson on the Ox 19
Dove on Storms 8
Dyer's City of Rome 2

Eastlake's Hints on Household Taste 12
Edwards's Shipmaster's Guide 20
Elements of Botany 9

Ellicott's Commentary on Ephesians 14
———— Destiny of the Creature 14
———— Lectures on Life of Christ 14
———— Commentary on Galatians 14
———————— Pastoral Epist. 14
———————— Philippians,&c. 14
———————— Thessalonians 14
Engel's Introduction to National Music .. 11
Essays and Reviews 15
——— on Religion and Literature, edited by Manning, First and Second Series .. 15
Ewald's History of Israel 14

Fairbairn's Application of Cast and Wrought Iron to Building 13
———— Information for Engineers 13
———— Treatise on Mills and Millwork 13
Fairbairn on Iron Shipbuilding 13
Farrar's Chapters on Language 5
Felkin on Hosiery & Lace Manufactures.. 13
Ffoulkes's Christendom's Divisions 15
Fliedner's (Pastor) Life.................. 4
Francis's Fishing Book 19
———— (Sir P.) Memoir and Journal 3
Froude's History of England 1
——— Short Studies 6

Ganot's Elementary Physics 8
Gilbert and Churchill's Dolomite Mountains 16
Gill's Papal Drama 2
Gilly's Shipwrecks of the Navy 17
Goodeve's Elements of Mechanism........ 13
Gorle's Questions on Browne's Exposition of the 39 Articles 14
Grant's Ethics of Aristotle................ 4
Graver Thoughts of a Country Parson...... 6
Gray's Anatomy.......................... 11
Greene's Corals and Sea Jellies 9
———— Sponges and Animalculae........ 9
Grove on Correlation of Physical Forces .. 8
Gwilt's Encyclopædia of Architecture 12

Handbook of Angling, by Ephemera...... 19
Hare on Election of Representatives 5
Harley and Brown's Histological Demonstrations 11
Hartwig's Harmonies of Nature.......... 9
———— Polar World 9
———— Sea and its Living Wonders.... 9
———— Tropical World................ 9
Haughton's Manual of Geology 8
Hawker's Instructions to Young Sportsmen 19
Hearn's Plutology 2
——— on English Government 2
Helps's Spanish Conquest in America 2
Henderson's Folk-Lore 6
Herschel's Essays from Reviews 10
———— Outlines of Astronomy........ 7
———— Preliminary Discourse on the Study of Natural Philosophy 8
Hewitt on the Diseases of Women 10
Hodgson's Time and Space................ 7
Holmes's System of Surgery 10
Hooker and Walker-Arnott's British Flora 9
Hopkins's Hawaii 8
Horne's Introduction to the Scriptures .. 14
——— Compendium of the Scriptures .. 14

Horsley's Manual of Poisons 9
Hoskyns's Occasional Essays.............. 7
How we Spent the Summer................ 16
Howard's Gymnastic Exercises 11
Howitt's Australian Discovery............ 16
——— Rural Life of England 17
——— Visits to Remarkable Places 17
Hudson's Executor's Guide 20
Hughes's Garden Architecture............ 13
———— (W.) Manual of Geography 7
Hullah's History of Modern Music 11
———— Transition Musical Lectures 11
———— Sacred Music 11
Humphreys's Sentiments of Shakspeare .. 12
Hutton's Studies in Parliament 6
Hymns from *Lyra Germanica* 14

Ingelow's Poems 18
——— Story of Doom 18
Icelandic Legends, Second Series 17

Jameson's Legends of the Saints and Martyrs 12
———— Legends of the Madonna 12
———— Legends of the Monastic Orders 12
Jameson and Eastlake's History of Our Lord 12
Jenner's Holy Child...................... 18
Johnston's Gazetteer, or General Geographical Dictionary 7

Kalisch's Commentary on the Bible...... 5
———— Hebrew Grammar.............. 5
Keith on Destiny of the World............ 14
———— Fulfilment of Prophecy.......... 14
Keller's Lake Dwellings of Switzerland .. 9
Kesteven's Domestic Medicine 11
Kirby and Spence's Entomology.......... 9
Knight's Arch of Titus.................... 17

Lady's Tour round Monte Rosa 16
Landon's (L. E. L.) Poetical Works 18
Latham's English Dictionary.............. 5
———— River Plate...................... 7
Lawrence on Rocks 8
Lecky's History of Rationalism 3
Leisure Hours in Town.................... 6
Lessons of Middle Age 6
Letters of Distinguished Musicians 4
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Liddell and Scott's Greek-English Lexicon 6
———————— Abridged ditto 6
Life of Man Symbolised.................... 12
Lindley and Moore's Treasury of Botany 9
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Loudon's Encyclopædia of Agriculture 13
———————— Gardening 13
———————— Plants 9
———————— Trees and Shrubs 9
——— Cottage, Farm, and Villa Architecture 13
Lowndes's Engineer's Handbook.......... 12
Lyra Domestica........................ 15
—— Eucharistica 16
—— Germanica 12, 16
—— Messianica 16
—— Mystica 16
—— Sacra 16
Macaulay's (Lord) Essays................ 2

Macaulay's History of England 1
—— Lays of Ancient Rome 18
—— Miscellaneous Writings 6
—— Speeches 5
—— Works 1
Macfarren's Lectures on Harmony 11
Macleod's Elements of Political Economy 4
—— Dictionary of Political Economy 4
—— Elements of Banking 19
—— Theory and Practice of Banking 19
McCulloch's Dictionary of Commerce 19
—— Geographical Dictionary 7
Maguire's Irish in America 17
—— Life of Father Mathew 3
—— Rome and its Rulers 3
Malleson's French in India 2
Manning on Holy Ghost 15
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Marshall's Physiology 10
Marshman's History of India 2
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Martineau's Endeavours after the Christian Life 16
Massey on Shakspeare's Sonnets 18
——'s History of England 1
Massingberd's History of the Reformation 3
Maunder's Biographical Treasury 4
—— Geographical Treasury 8
—— Historical Treasury 3
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May's Constitutional History of England 1
Melville's Digby Grand 17
—— General Bounce 17
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—— Good for Nothing 17
—— Holmby House 17
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—— on Representative Government 4
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—— His Presence not his Memory 15
—— 'Spiritual Songs' 15
Montgomery on Pregnancy 10
Moore's Irish Melodies 18
—— Lalla Rookh 18
—— Journal and Correspondence 3
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—— (Dr. G.) First Man 8
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Pictures in Tyrol 16
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—— Chemical, Natural, and Physical Magic 13
Pike's English and their Origin 6
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—— Glimpse of the World 16
—— History of the Early Church 3
—— Journal of a Home Life 17

Sewell's Passing Thoughts on Religion .. 15
——— Preparation for Communion 15
——— Principles of Education 15
——— Readings for Confirmation 15
——— Readings for Lent 15
——— Examination for Confirmation .. 15
——— Stories and Tales 17
Shaw's Work on Wine 20
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